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HIGHWAY RESEARCH REPORT

LABORATORY CORROSION TEST OF STEEL IN CONCRETE

INTERIM REPORT

68-15

STATE OF CALIFORNIA

TRANSPORTATION AGENCY

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

MATERIALS AND RESEARCH DEPA[illegible]

RESEARCH REPORT

NO. M & R 635116-3

STATE OF CALIFORNIA—TRANSPORTATION AGENCY

RONALD REAGAN, Governor

DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT
5900 FOLSOM BLVD., SACRAMENTO 95819

September, 1968
Interim Report
M&R No. 635116-3
Federal No. D-3-11

MR. J. A. LEGARRA
State Highway Engineer

Dear Sir:

Submitted herewith is a research report titled:

Laboratory Corrosion Test of Steel
in Concrete

Donald L. Spellman
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Very truly yours,

JOHN L. BEATON
Materials and Research Engr.

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Abstract: A method for laboratory corrosion testing of steel in concrete was correlated to previous laboratory and field investigations. Included in the report are derived mathematical relationships of the influence of moist or steam curing, admixtures, mixing water, entrained air, and cement content of concrete to the time of an active half-cell potential of the embedded steel. Some statistical data are presented which indicate the numerical value which demarks the upper limit of a passive half-cell potential of the steel. Other data show the measured corrosion loss of the steel and the quantity of chloride found in concrete fragments. Also presented is what is believed a new method for applying a regression analysis to the distribution of test values.

Key Words: Corrosion, concretes, steel, statistics, testing, test methods, admixtures, electrical potential, curing, entrained air, cement content, chloride, laboratory tests, field tests, mixing water, cement factor, regression analysis.

ACKNOWLEDGEMENT

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The opinions, findings, and conclusions expressed in this report are those of the authors and are not necessarily those held by the Bureau of Public Roads.

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LABORATORY CORROSION TEST OF STEEL IN CONCRETE

INTRODUCTION

Beginning in 1957[1], the California Division of Highways has been reporting on the causes of corrosion of steel in concrete. In 1963, an empirical equation[2] was developed from field and laboratory data for estimating the time to corrosion of embedded steel. This previous work indicated that additional data were required regarding the influence of curing, admixtures, and other concrete design variables before designing a structure for a specific maintenance-free life[2].

Based upon a literature survey, it was found that at least 15 different methods have been used for testing the corrosion of steel in concrete[3]. In general, some of the methods are:

1. Exposure to tidal water[4,5]
2. Normal outdoors[5,6,7,8,9,10]
3. Laboratory, high humidity[6,11,8,12]
4. Laboratory, low humidity[6,13]
5. Alternate immersion[6,12]
6. Alternate but partial immersion[14,15]
7. Variable salt, moisture, temperature[16]
8. Salt spray cabinet[17]
9. Partially covered with a wet towel[13]
10. Flow of water vapor[18]
11. Periodic spraying with salt water[18]
12. Immersed in water[12]
13. Partial immersion[9]
14. Dry cellar[19]
15. Impressed voltages[1]

In this investigation, it was considered that an appropriate test method should be one that is similar to field conditions. That is, the test should consist of the partial immersion of reinforced concrete in a salt solution.

In effect, this research project was to not only determine the influence of some variables of concrete manufacture, but to also explore what was thought to be a reasonable and appropriate means for the corrosion testing of reinforced concrete.

Reported herein are the data from the completed half-cell potential measurements and the available and pertinent concrete cracking data. Currently, not all of the reinforced concrete has cracked as a result of corrosion. Therefore, when the concrete cracking phase of the test program is completed, it will be reported.

SUMMARY AND CONCLUSIONS

For this phase of the data analysis, it appears that the test results obtained from the partial immersion of reinforced concrete in a saturated solution of sodium chloride can be related to the probable performance of bridge substructures in California environments.

The test data indicate that there is a definite mathematical relationship between the time to an active potential and the time to cracking of the concrete as a result of the corrosion of the steel. Therefore, the length of testing time can be reduced if the criterion for the conclusion of a test result is the time to an active potential rather than the more time-consuming time to visible evidence of distress.

These test results indicate that the time to corrosion of reinforcing steel is controlled by the chloride content of the environment, the gross mixing water and cement content of the concrete, and the concrete curing method. In general, it was found that the time to an active corrosion potential was increased by (a) reducing the mixing water, and (b) increasing the cement content. Conversely, the time to an active corrosion potential was decreased by (a) steam curing, (b) increasing the mixing water, and (c) reducing the cement content.

The test results also indicate that the water reducing-set retarders and an air-entraining agent can be beneficial when there is a reduction in the amount of mixing water of the concrete. However, the data also indicate that an increase in the air content of freshly mixed concrete could result in a decrease in the time to an active half-cell potential. It did not appear to be highly significant as to whether or not the steel was oriented in a horizontal or vertical position when the concrete was placed.

There was no deliberate attempt to create air voids at the interface of the steel and concrete during fabrication of the test specimens. Therefore, it is apparent that either significant air voids are always present at the steel-concrete interface, or, it may be a simple fact that if the void system in concrete is sufficiently large to permit the passage of moisture and the chloride-ion to the surface of the embedded steel, then it is large enough to permit the corrosion of the steel.

The current data analysis indicates that the most probable maximum value for a passive half-cell potential of steel in aged concrete is about 200 millivolts negative to a saturated calomel electrode.

FABRICATION OF TEST SPECIMENS

The variables of concrete manufacture as used in this test are shown in Table 1. The river run aggregate was 3/4-inch maximum and the gradation complied with the 1964 Standard Specifications of the California Division of Highways. The cement used was Type II, Modified, low-alkali which also complied with the California specifications.

The reinforced concrete specimens were 4½-inches wide, 2½-inches thick, and 15 inches long. The No. 4 reinforcing steel was sandblasted and was cast in the concrete to provide a minimum of 7/8-inch concrete cover at any point.

Ten test blocks were fabricated from each of 20 batches. Half were moist cured, and half were steam cured. The moist cured specimens were cured in a fog room at 73°F for 7 days. The steam cured specimens were first steam cured for approximately 18 hours at a temperature range of 140° to 155°F, then moist cured for six days in a fog room.

After the moist and steam-moist curing process, all specimens were stored in laboratory air at approximately 72F and 50% relative humidity for approximately 10.5 months.

One-half, or 100 blocks, were cast with the steel in a horizontal position and in the other half, the blocks were cast with the steel in a vertical direction.

The amount of concrete placement energy could be considered about equal for both types of casting methods. The vibration of the concrete was accomplished by placing the wooden molds on a "Packer" type vibrating table. The only known difference in the methods of concrete placement for the two types of casting was that the concrete was "rodded" around the steel in the horizontally cast specimens with a trowel.

The concrete design slump was either two or four inches and was obtained by varying the quantity of mixing water.

The three types of admixtures used in this study were (1) a hydroxylated carboxylic acid which was added at a rate of 3.5 fluid ounces per sack of cement, (2) a lignosulfonate type which was added at the dosage of 0.20-pound per sack of cement. When these two types of admixtures were used, the mixing water was reduced so as to maintain the desired concrete slump. Both admixtures conformed to the ASTM Specification C-494, Type D, Water Reducing and Set Retarding Admixtures.

The third admixture used was a neutralized Vinsol resin air entraining agent which was added at a dosage of 0.75 fluid ounces per sack of cement to entrain approximately 4% air.

In this study, one set of five specimens were used to test each variable.

TESTING PROCEDURE

The first phase of corrosion research procedure was to expose the reinforced concrete blocks to water that was essentially free of chloride.

For approximately 5½ months, the test prisms were partially immersed to varying depths of fresh water in a vertical position (see Figure 1). The purpose of this type of exposure was to determine the half-cell potential of the steel in an environment which is known to be non-aggresive. In Figure 1, a half-cell potential of about 360 MV is shown at the one month period of exposure. This could be the result of wetting the steel with fresh water at the interface where the steel and concrete are both in contact with the atmosphere. Therefore, these potential readings are probably representative of steel in fresh water rather than one of steel embedded in concrete.

In general, the half-cell potential of the steel was measured thrice weekly with a data acquisition system of 10 meghoms input impedance, or manually with a voltmeter of $1x10^{10}$ ohms impedance.

The standard reference cell used in this study was a saturated calomel electrode.

Immediately after the fresh water test procedure, the water level was lowered from 12 inches to 3½ inches and the solution was then saturated with sodium chloride.

In general, the test program during this second phase, while the reinforced concrete was partially immersed in 3½ inches of brine, was as follows:

A. Half-cell potentials of the steel to a saturated calomel half-cell were taken thrice weekly.

B. Whenever there was a measured significant change in the half-cell potential of the steel, thereafter the surface of the block was normally inspected once every ten days for visual evidence of corrosion. Visual evidence of corrosion was considered to be the observation of a rust stain or concrete crack. The visual observations of the concrete surface was made with the assistance of a three power magnifying glass.

C. After approximately one year and eight months, the salt encrustations on specimens, which had no measured significant change in potential, were washed off and visually checked for evidence of corrosion.

D. After approximately two years and ten months of exposure, salt encrustations were mechanically brushed from the surface of the concrete specimen which had no measured significant change in half-cell potential.

E. When a complete set of five specimens were visually observed to have evidence of corrosion, they were removed from the test and subjected to the following procedure:

a) The concrete was thoroughly washed to remove abnormal concentrations of salt on the surface.

b) Samples of concrete were removed and subjected to chemical analysis for chloride content (1) from the top of the block to 4 inches below the top; (2) from 4 inches to 8 inches below the top; (3) from 8 inches to 12 inches below the top; and (4) from 12 inches below the top to the bottom of the specimen. (Keep in mind that only the bottom 3½ inches were immersed in salt water.)

c) After removal from the concrete, the reinforcing steel was chemically cleaned in a solution of sulfamic acid and a propriety corrosion inhibitor.

d) After the steel was cleaned, maximum depths of metal loss were determined by means of a micrometer.

e) Beginning with the removal of the concrete specimens from test, a photographic record was made of the visible evidence of corrosion on the surface of the concrete. Then, when the steel was exposed, a record was made of the appearance and location of rust on the steel and concrete surfaces.

TEST RESULTS

A. Potential Measurements

As indicated by Figure 1, Typical Potential Measurement versus Time, the half-cell potential of steel in concrete is generally less negative than about 150 millivolts (MV) to a saturated calomel electrode (SCE) in a fresh water exposure. When the concrete is partially immersed in an apparently saturated sodium chloride solution (approximately 160,000 ppm chloride), the potential is generally less negative than about 175 MV-SCE.

Table 2 is a tabulation of all of the current test results for the reinforced concrete blocks.

In Figure 1, at about the 7½ total month exposure period, there is an abrupt change from an apparently passive potential of approximately -150 MV to an active potential of -350 MV-SCE. On all 200 specimens, an abrupt change from a passive to an apparently active potential such as this was observed. For the three specimens on which a continuous potential recording was made, the change to an active potential occurred during an approximately 3½ hour interval. As noted in Figure 1, the concrete was observed to be cracked as a result of the corrosion of the steel at a total test time of approximately 10 months. Figure 2, Distribution of Potentials, is a plot of the half-cell potential of all 200 specimens at four different times.

As indicated by the distribution curves shown on Figure 2, it appears that the most probable maximum value for a passive half-cell potential of steel in aged concrete is approximately -200 MV-SCE. The difference between the potentials shown for the fresh water and brine are most likely the difference in the solution pressure of the electrode itself. Also, the exposure which is indicated to be fresh water may more likely be termed alkaline water because of the leeching of the alkaline products from the concrete blocks.

In the periodic measurements, it was found that in some cases the steel with half-cell potentials in the order of -250 MV-SCE would not only fluctuate but reduce to the level of an apparently passive potential. Further investigation indicated that some of these apparently low level initial active potentials reduced to values that were in the range of -100 MV to -175 MV. The current data indicate that in about 12 percent of the blocks the potential decreased for a period of time. However, the potential would always become more negative at a later date.

In general, it was observed that passive potentials fluctuated with time, but active potentials not only fluctuated but generally increased in value with time. The data shown in Figure 2 shows that the mean active potential for the 200 specimens was -330 MV on the first day of a measured potential change as compared to -440 MV after one additional week of exposure.

B. Potential and Concrete Cracking

This series of tests is not completed because not all of the concrete specimens have cracked as a result of corrosion of the steel. However, there are currently 34 sets of five blocks which both have an apparent active potential and a resultant crack in the concrete. Therefore, if the abrupt change in the potential is a meaningful value, there should be a direct relationship between the time to the potential change and the time to concrete cracking. For this determination the "days to active potential" is the average time that it took a set of five blocks to reach an active potential while partially immersed in the brine. The "average days to concrete cracking" was the average time for the concrete to have an observable crack on its surface while partially immersed in brine.

As shown in Figure 3, Concrete Cracking versus Active Potential, the derived relationship was as follows:

$$C = 1.12\,P + 115 \qquad (1)$$

Wherein:

C = Average days to concrete cracking for 5 blocks

P = Average days to an active potential for 5 blocks

For this equation, the correlation coefficient was 0.877, the standard error of estimate was 75 days, and the number of data points were 34 sets of five blocks.

Equation (1) indicates that there is a definite relationship between the time to the abrupt change to the apparently active half-cell potential of the steel and the time to the visual evidence of concrete cracking. The inspection of concrete surfaces for evidence of cracking or rust stains is not only time consuming, but it prolongs the testing program over that when only considering potential measurements. Therefore, since the time to an active potential is a meaningful test criterion, future testing may not deliberately include the observations for concrete cracking on specimens of the same dimensions.

C. Effect of Concrete Curing

As mentioned previously, one-half of the specimens were moist cured and half were steam cured. Because the concrete mixes were otherwise identical, the "average days to an active potential" for each type of curing was examined for a mathematical relationship by the method of least squares.

As shown in Figure 4, Average Days to Active Potential of Steam versus Moist Curing, the mathematical relationship obtained was as follows:

$$P_s = 0.553\ P_m - 3.63 \quad \text{. . . . .} \quad (2)$$

Wherein:

P_s = Average days to active potential for 5 steam cured blocks

P_m = Average days to active potential for 5 moist cured blocks

The significance of the derived equation is indicated to be at a high level as evidenced by a correlation coefficient of 0.939, and a standard error of the estimate of 38 days for the 20 comparative data points. In total, the 20 data points shown in Figure 4 represents the test results of 200 test specimens.

It is apparent that the steam curing of concrete is not beneficial with regard to its effect on protecting the steel from corrosion. In effect, steel in steam cured concrete in an aggressive environment will begin corroding in about 55 percent of the time that it will take steel to corrode in moist cured concrete. It should be pointed out, however, that much of the concrete which is steam cured is made with a high cement factor and a low water-cement ratio which would offset some of the reduced time to corrosion when using leaner mixes, and therefore, the time to corrosion may still result in an economically acceptable life.

D. Effect of Concrete Mixes

Figures 5 and 6 graphically show the effect of the various concrete mixes and the two methods of concrete curing. The averaged data shown in these figures were limited to the results obtained from the 4-inch nominal slump concrete.

As shown in Figures 5 and 6, the two important variables which seem to affect the time to corrosion are the method of concrete curing and the cement factor. The influence of the admixtures appears to be most significant as shown for the 8-sack concrete mix in Figure 6.

In these figures, the admixture designated as "VR" is a Vinsol resin air-entraining agent; "L.S." is a lignosulfonate type; and "H.C.A." is a hydroxylated carboxylated acid type admixture. The latter two concrete admixtures are classified as water-reducing and set-retarding admixtures.

In order to determine the influence of the concrete mixes to the average time of an active potential for each set of five concrete blocks, the data were subjected to a multiple regression analysis by the method of least squares. The mathematical analysis resulted in the following equation:

$$D = \frac{K_1 \, C^{K_2}}{W^{K_3} \, A^{K_4}} \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

Wherein:

D = Average days to active potential for 5 blocks partially immersed in a chloride solution

K_1 = Equation constant

C = Cement factor in sacks per cubic yard

W = Mixing water in percent of concrete volume

A = Entrained plus entrapped air - percent

K_2, K_3, K_4 = Exponent of the variable

Because of the multiple number and value of the derived equation variables, the results are tabulated in Table 3, Influence of Concrete Mix Variables on Time to Active Potential.

As indicated by the mathematical analysis, the influence of the admixtures on the time to active potential of the steel could be explained by their ability to reduce the concrete mixing water and by this factor appears to prolong the time to corrosion. It is also of interest to note that entrained air per se does not appear to be beneficial with regard to the time to an active potential of the steel. However, it should be noted that the available data is relatively limited.

It must be emphasized that the importance of entrained air in inhibiting the deterioration of concrete as a result of freezing and thawing is in no way impugned. The corrosion of embedded steel cannot be prevented when the concrete is deteriorated by the action of freezing and thawing. Also, it will be noted that Equation (3) included a factor for mixing water. The proper use of an air-entraining agent results in a reduction of the mixing water requirement. Therefore, even in a corrosion sense, the air entrainment of a concrete mix can be self-balancing or even beneficial when considering mixing water reduction, workability and resistance to freezing and thawing.

In Equation (3) it is observed that the mixing water and cement contents of the concrete are of primary importance in controlling the time to an active potential of the steel.

It is also of interest to note that mathematically, Equation (3) indicates that if the amount of entrained and entrapped air goes to zero, the time to corrosion is infinite. Obviously this facet of the data should be vigorously investigated to establish the limits and relationships between air entrainment and the time to corrosion.

E. Metal Loss

After the reinforcing steel was removed from the concrete and was cleaned with the inhibited acid, the deepest corrosion penetration into the steel was measured to the nearest one-thousandth of an inch with a micrometer.

Thus far, not all of the reinforced concrete specimens have been removed from test because not all have cracked as a result of steel corrosion. However, it appears that sufficient data are available wherein a reasonable description can be made of the current results of metal loss.

Figure 7, Distribution of Metal Loss, shows that the current mean value of metal loss that was measured was about ten one-thousandth of an inch. This reported metal loss is only for those samples which were removed from test in less than two weeks after the visual observation of a crack or rust stain on the concrete surface. Longer exposure, of course, would be expected to increase metal loss.

In Figure 7, it will be noted that in about 16% of the cases, a metal penetration of one-one thousandth of an inch or less was sufficient to result in concrete cracking and/or rust stains.

In Figure 8, Distribution of Corrosion Rate, the mean corrosion rate is approximately 35 mils per year, while the maximum was about 138 mils per year. For the 41 observations, the total corrosion time was calculated from the time when an active potential was first measured to not later than two weeks from the time when a crack was observed on the surface of the concrete.

The mean values reported for Figures 7 and 8 are on the basis of a visual "best fit" of the data which is plotted on probability paper.

By the method of least squares, the 41 measurements of corrosion loss were compared to the active corrosion time. The resulting equation was linear and is as follows:

$$M = 16.8Y + 4.39 \quad \ldots \quad (4)$$

Wherein:

M = Maximum pit depth in mils at time of cracking

Y = Time in years from active potential to concrete cracking

For this relationship, the correlation coefficient was 0.457 and the standard error of the estimate was 8.78 mils.

In conjunction with the maximum pit depth measurements, the surface area of rust on the steel was visually estimated. By the method of least squares, the 41 observations were subject to a multiple regression analysis. The "best fit" of the data were in linear terms and the equation was:

$$M = 17.88Y - 0.30A + 5.46 \quad \ldots \quad (5)$$

Wherein:

M = Maximum pit depth in mils at time of cracking

Y = Time in years from active potential to concrete cracking

A = Area of the rust, in percent of embedded steel surface

The correlation coefficient was 0.529, while the standard error of the estimate was 8.38 mils.

As indicated by this equation, the larger the rusted area, the less the maximum depth of metal loss. It is apparent that the corrosion rate of steel in concrete is related to the relative surface area of the anode (the rusted area), and the cathode (the nonrusted area). From this, it would be expected that progressive rusting over the surface area of the steel should result in a reduction in the maximum pit depth with time. However, it also appears that this benefit could be marginal if the rusting results in continual concrete spalling and the direct exposure to corrosives. Also, these data do not reflect the effect of a corrosion-caused concrete crack on the continued corrosion rate of the steel.

F. Chloride in Concrete

As the concrete specimens were removed from their 3½ inch depth of partial immersion in the brine solution, samples of the concrete were removed and chemically analyzed for chloride content by means of an acid titration.

The results of the chemical analysis are shown in Figure 9, Distribution of Chloride in Concrete Fragments. As indicated in Figure 9, the chloride content of the samples which have been currently analyzed is not constant at any location on the concrete.

It is of interest that the concrete specimen noted on Figure 9 as 0" - 4", which was located a minimum of 7½" above the surface of the brine solution, had an indicated mean calculated chloride content of about one pound per cubic yard of concrete. For the concrete which was totally immersed in the brine, the indicated mean calculated chloride content was about 27 pounds per cubic yard.

G. Orientation of Steel

As previously noted, one-half of the test specimens were cast with the steel in a horizontal and one-half with the steel in a vertical position.

Because the concrete used in all the horizontal or vertical specimens did not come from one batch, there may be differences in the test results because of variations in cement factor, mixing water, etc.

For example, the average cement factor for the horizontally cast steel specimens was 7.08 sacks of cement, while the vertically cast specimens had 7.18 sacks of cement per cubic yard of concrete. Based upon the difference in cement factor alone, it would be expected that the vertically cast steel would have a longer test time to an active potential. Such was the case. The average time to an active potential for the vertically cast steel was 272 days, while the horizontally cast steel was 253 days.

By the method of least squares, the following equation was derived to show the relationship between the orientation methods for placement of the steel in comparable concrete:

$$D_v = 0.988\ D_h + 22 \quad \ldots \ldots \ldots \ldots \ldots \quad (6)$$

Wherein:

D_v = Average days to an active potential for a set of 5 samples, steel cast vertically

D_h = Average days to an active potential for a set of 5 samples, steel cast horizontally

The coefficient of correlation for Equation (6) was 0.867 and the standard error of the estimate was 93 days for the 20 data points.

As indicated by Equation (6), the effect of the orientation of the steel to the time to an active potential appears to be relatively minor.

H. Test Method Variation

Normally, when an unproven test method is employed, it is desirable to (1) determine the accuracy of the test in reflecting a field problem, (2) determine the reproducibility of the test itself, and (3) determine the influence of the number of replicate samples on the apparent accuracy of the test results.

A mathematical relationship was derived between the mean values of a set of samples and the consecutive order of the failure of the individual sample in a set.

An example of this is shown in Figure 10, Days to Active Potential of Third Block versus Set of Five. In this figure, the days to the active potential for the third consecutive block is plotted against the mean days to active potential for the entire set of five.

The relationship of the third block versus the average time to the active potential for the set of five is shown in Figure 10. The following equation was derived to express the relationship:

$$P_3 = 1.11\ P_a - 25 \ldots \ldots \ldots \ldots \ldots \quad (7)$$

Wherein:

P_3 = Days to active potential of third consecutive block

P_a = Days to active potential for all five blocks

The significance of the regression analysis is indicated by the correlation coefficient of 0.982, and a standard error of estimate of 36 days for the 40 single blocks versus the mean of 40 sets of five samples.

This method of analysis was applied to the test results of each block for each consecutive time to active potential versus the mean for the set of five.

The results of the regression analysis for all 5 blocks to consecutively reach an active potential is shown in Figure 11, Days to Active Potential of Nth Block versus Set of Five. As indicated by Figure 11, the variance in the performance of individual samples can be tempered by the overall relationship between the average versus the consecutive failure time of each individual block.

By using the computed regression lines as shown in Figure 11, the standard deviation was calculated by normal means and the results are shown in Figure 12.

From the derived relationship between the standard deviation versus the mean days to active potential as shown in Figure 12, the coefficients of variation of the data were calculated and are plotted in Figure 13, Coefficient of Variation versus Active Potential.

As shown in Figure 13, the coefficient of variation can be about 60 percent when the average days to an active potential is about 50 days. At approximately 300 average test days, the coefficient of variation of the test results reduces to about 33 percent and practically remains constant.

As an indication of the significance of the coefficient of variation, the maximum error of the mean versus the number of test specimens was calculated[23] at the 95 percent confidence level for three coefficients of variation and are shown in Figure 14. It is interesting to note that with five samples and a coefficient of variation of 60 percent, the maximum error of the mean at the 95 percent confidence level could be as great as about 52 percent. Conversely, with a coefficient of variation of 60 percent and 500 specimens, the expected maximum error of the mean could be about five percent at the 95 percent confidence level.

It therefore appears that due consideration must be given not only to the number of specimens in the corrosion testing of steel in concrete, but a great deal of emphasis should be given to the testing time. Otherwise the significance of variations in test data may only be due to random chance instead of a real test variable.

I. Correlation of the Test Method

Bridge substructures have an environmental exposure of partial immersion or burial. Therefore, it would be consistent to expose laboratory corrosion test specimens in the same manner if the data are to be related to bridges. In an attempt to relate the laboratory test to the performance of field structures and other laboratory tests, the time to corrosion of the moist cured specimens having access to chlorides was calculated by means of the following equation[2]:

$$R = \frac{1.107^{C}\, C^{0.717}\, S^{1.22}\, 1011}{K^{0.42}\, W^{1.17}} \quad \ldots\ldots\ldots \quad (8)$$

Wherein:

R = Years to visible evidence of reinforcing steel corrosion

C = Sacks of cement per cubic yard of concrete

S = Depth of concrete cover over reinforcing steel - inches

K = Parts per million of chloride-ion in the environment

W = Gross mixing water in percent of concrete volume and includes that contained by aggregate

On the average, the actual time for the moist cured test specimens to reach an active potential was 341 days, while the calculated time to visible evidence of corrosion by means of Equation (8) would be 442 days. Based upon the relatively close agreement between the actual and calculated testing times, it appears that the results of this test method could be related to other laboratory and field data[2].

By the method of least squares, regression lines were calculated to determine the mathematical relationship between the calculated time to corrosion and the actual time to active potential of the moist and steam cured reinforced concrete specimens. The following relationships were derived:

$$A_m = 0.000809C^{2.11} \qquad (9)$$

Wherein:

A_m = Actual average days to an active potential for 5 moist cured specimens

C = Days to corrosion as calculated by Equation (8)

For the twenty data points, the correlation coefficient was 0.908 and the standard error of estimate is the multiplier or divisor of 1.251.

For the steam cured concrete, the correlation was:

$$A_s = 0.000318C^{2.16} \qquad (10)$$

Wherein:

A_s = Actual average days to an active potential for 5 steam cured specimens

C = Days to corrosion as calculated by Equation (8)

For the twenty data points, the correlation coefficient was 0.888 and the standard error of estimate is the multiplier or divisor of 1.294.

As indicated by derived relationships shown in Equations (9) and (10), it is apparent that the results from the partial immersion corrosion testing of reinforced concrete can be mathematically related to other laboratory and field experiences.

DISCUSSION

The partial immersion of reinforced concrete in a saturated solution of sodium chloride shows promise as a laboratory test method which can be mathematically related to the anticipated performance of full scale bridge substructures in California. As indicated by the variations in test times of apparently identical specimens, it is obvious that the number of samples that are required for a meaningful test result will far exceed two or three samples.

Because these test results have indicated that the times to failure are normally distributed, consideration is given to the feasibility of fabricating a large number of replicate specimens. With a large number of test specimens, it appears that if the test were terminated when a significant number or percentage of samples have failed instead of all of them, then the remainder of the distribution curve could be extrapolated. Therefore, even though there is a greater initial cost for fabricating a larger number of samples, it is quite possible that testing could be reduced by finishing in a more reasonable length of time rather than waiting for the last one to fail. Statistically, the length of time for the last sample to fail could be quite long.

One other factor that has not been directly evaluated in the testing of the reinforced concrete is the ambient moisture content immediately prior to its contact with the corrosive solution. In this test the time to concrete cracking due to corrosion of the steel in six-sack moist cured concrete which was first exposed to fresh water, averaged about ten months. In a previous test[3] where almost identical specimens were first exposed to the drying action of laboratory air for 60 days, and then immediately subjected to brine, the time to cracking was about one month. Therefore, it seems highly probable that corrosion test results are highly affected by the following variables:

1. Number of samples
2. Initial and ambient moisture content of the concrete
3. Temperature
4. Relative humidity
5. Salinity of the solution

6. The surface area to volume of the sample

7. Concrete manufacture variables

8. Concrete cover over the steel.

From the data thus far obtained, it appears that the measurements of the change in the half-cell potential of the steel from a passive to an active value is a significant test criterion. It is possible that the half-cell potential may be a more definitive method of evaluation because it would not be dependent upon concrete strength and other factors as in the criterion of concrete cracking. Also, the inspection of the concrete surface for cracks is neither an easy nor welcome task.

One other benefit of using a test criterion of the time to change in the potential of the steel is that the length of testing time can be significantly reduced. For example, the average time to cracking for the 6-sack moist cure, no admixture concrete was about 10 months, while the average time to an active potential was about 6 months.

A regression analysis of the times to consecutive failure is presented. At this time, we have not thoroughly investigated the mathematical technique. However, the method may be of further value in the analysis of possible changes in the shape of distribution curves for various types of data. Also, prior to the use of this method of analysis, the distribution of the coefficients of variation for the individual data as normally calculated, were so scattered that they only presented a "hint" that they varied with the time of test. It is anticipated that this method of data analysis will receive further review.

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TABLE 1

Concrete Mixes

Admixture		Steel Position When Cast	Percent by Volume		Fineness Modulus	Mix Water % by Volume		Slump, Ins.	% Air	Cement Factor
Type	per sack		Rock	Sand		Gross	Net			
None	---	Vertical	50	50	4.96	21.85	19.06	2¼	2	5.97
None	---		50	50	4.96	22.70	19.97	4	1.8	6.01
No. 1	3.50 oz.		50	50	4.96	20.46	17.61	4	2.4	6.11
AEA	3/4 oz.		52	48	5.02	21.41	18.73	4¼	4.3	6.08
No. 2	0.20 lb.		51	49	4.98	20.24	17.45	3-3/4	2.9	6.14
None	---		54	46	5.09	22.18	19.62	2¼	1.7	8.28
None	---		54	46	5.09	23.03	20.51	4½	1.4	8.36
No. 1	3.62 oz.		54	46	5.09	20.44	17.80	4	2.2	8.26
AEA	3/4 oz.		56	44	5.15	22.39	19.95	4½	4.0	8.30
No. 2	0.20 lb.		55	45	5.12	20.58	18.03	4	2.7	8.26
None	---	Horizontal	50	50	4.96	20.94	18.11	2	1.8	6.06
None	---		50	50	4.96	22.41	19.67	4½	1.5	6.04
No. 1	3.50 oz.		50	50	4.96	20.52	17.67	4	1.8	6.01
AEA	1.05 oz.		52	48	5.02	20.11	17.40	4	4.1	6.09
No. 2	0.20 lb.		51	49	4.98	20.36	17.56	3-3/4	2.4	6.03
None	---		54	46	5.09	21.19	18.58	2½	1.7	8.11
None	---		54	46	5.09	21.66	19.09	4½	1.6	8.19
No. 1	3.51 oz.		54	46	5.09	20.39	17.75	4	1.9	8.01
AEA	1.08 oz.		56	44	5.15	21.04	18.56	4	3.6	8.15
No. 2	0.20 lb.		55	45	5.12	20.23	17.62	4	2.2	8.15

No. 1 - Hydroxylated carboxylic acid
No. 2 - Lignosulfonate
AEA - Neutralized Vinsol resin air entraining agent
Cement Factor - 94 pound sacks of cement per cubic yard of concrete.

Current Test Results

Cement Factor	Slump Ins.	Admixture	Curing	Days to Active Potential* Vertical	Days to Active Potential* Horizontal	Days to Concrete Cracking* Vertical	Days to Concrete Cracking* Horizontal
6 Sacks Cu. Yd.	2	None	Moist	219.0	177.2	403.8	359.8
			Steam	117.4	120.4	200.6	284.4
	4	None	Moist	228.4	118.0	382.2	237.8
			Steam	93.0	80.4	165.4	158.6
		No. 1	Moist	244.2	164.2	735.0	279.2
			Steam	148.6	100.2	262.2	214.4
		AEA	Moist	199.8	174.6	345.6	372.6
			Steam	85.4	118.2	162.8	232.6
		No. 2	Moist	246.6	190.6	409.8	317.2
			Steam	86.6	86.4	197.0	160.6
8 Sacks per Cu. Yd.	2	None	Moist	348.6	436.8	390.8	
			Steam	212.4	162.0	341.2	281.2
	4	None	Moist	339.0	399.2	406.8	
			Steam	174.8	263.8	296.8	396.4
		No. 1	Moist	798.6	624.8		
			Steam	406.2	326.2	588.0	499.8
		AEA	Moist	324.4	485.4	398.0	
			Steam	143.0	268.2	279.8	449.8
		No. 2	Moist	599.4	491.6	733.2	665.0
			Steam	423.6	274.6		495.6

* Average days for 5 samples
AEA - Air entraining agent: Neutralized Vinsol resin
No. 1 - Water reducing set retarder: Hydroxilated carboxylic acid
No. 2 - Water reducing set retarder: Lignosulfonate.

TABLE 3

Influence of Concrete Mix Variables on Time to Active Potential

Type of Cure	K_1	K_2	K_3 Gross	K_3 Net	K_4	Coef. of Correlation	Standard Error*
Moist	48,250	3.00	3.58			0.9134	1.2432
	5,164	3.12		3.06		0.9143	1.2418
	65,200	3.00	3.67		0.032	0.9136	1.2429
	5,539	3.12		3.08	0.010	0.9144	1.2417
Steam	96,436	3.06	4.06			0.8968	1.2811
	8,097	3.19		3.48		0.8986	1.2785
	548,567	3.04	4.56		0.184	0.9035	1.2713
	23,483	3.18		3.80	0.156	0.9035	1.2712

*The standard error of estimate is a multiplier and a divisor.

Figure 1

TYPICAL POTENTIAL MEASURMENTS VERSUS TIME

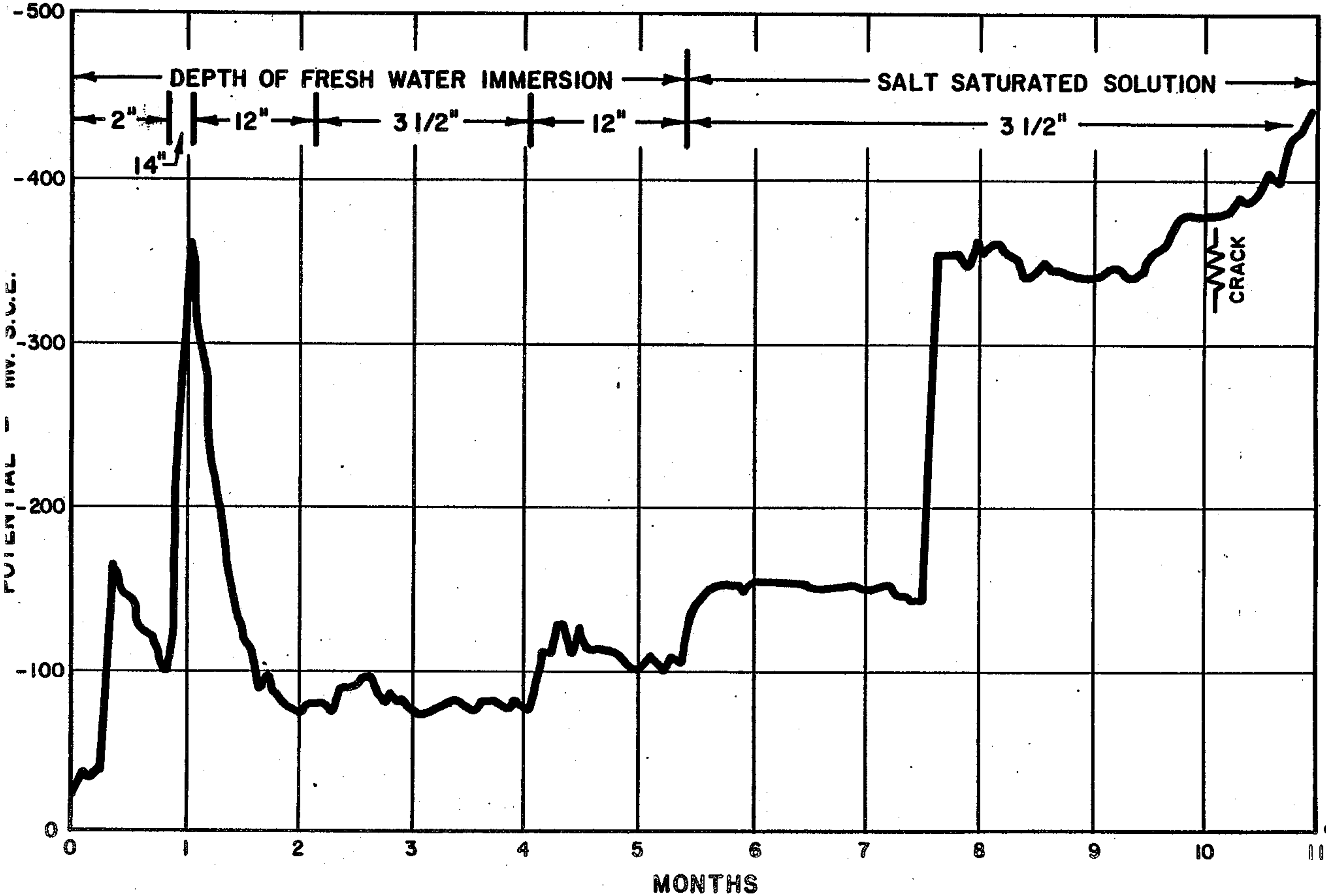
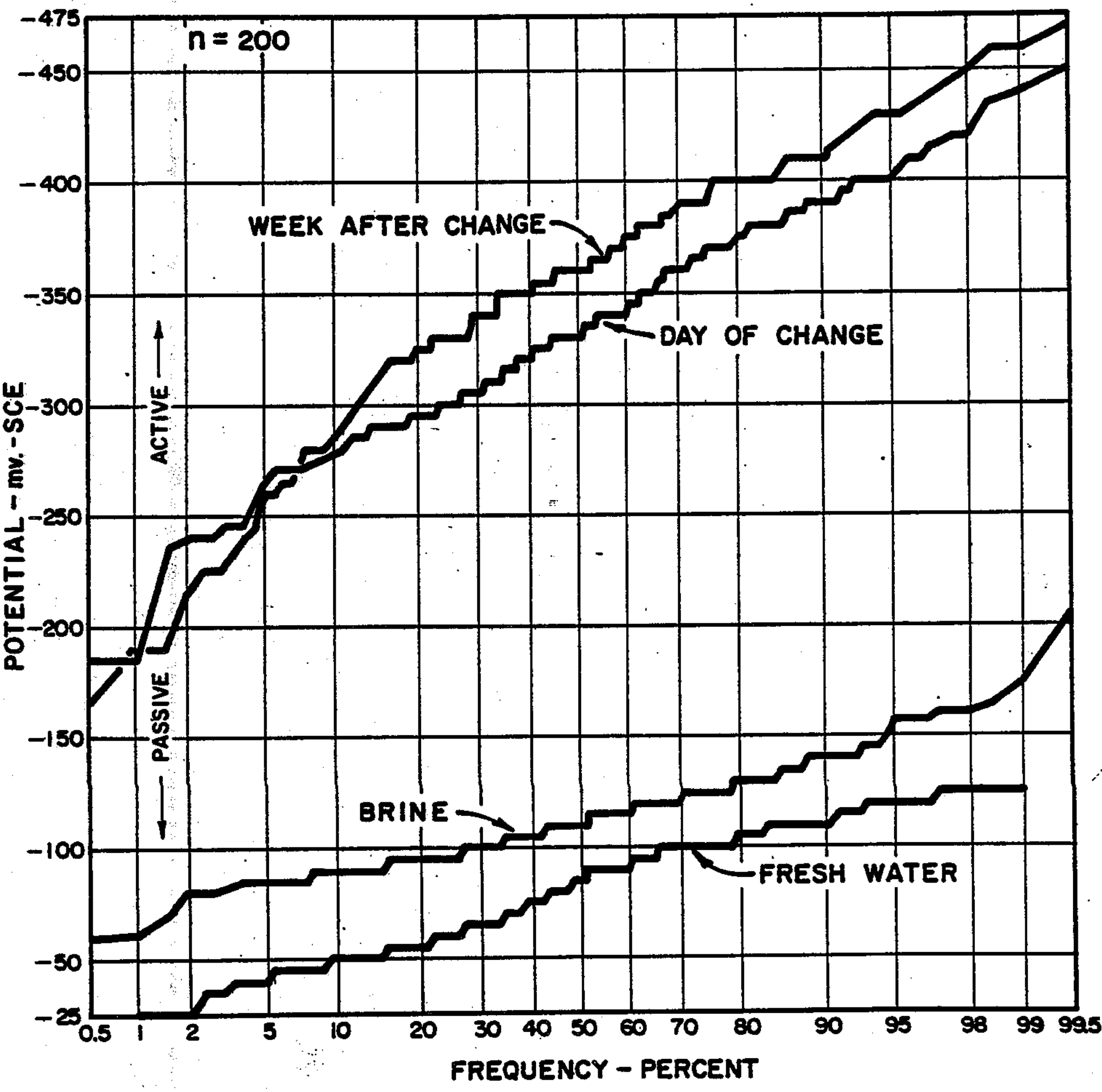


Figure 2

DISTRIBUTION OF POTENTIALS



CONCRETE CRACKING VERSUS ACTIVE POTENTIAL

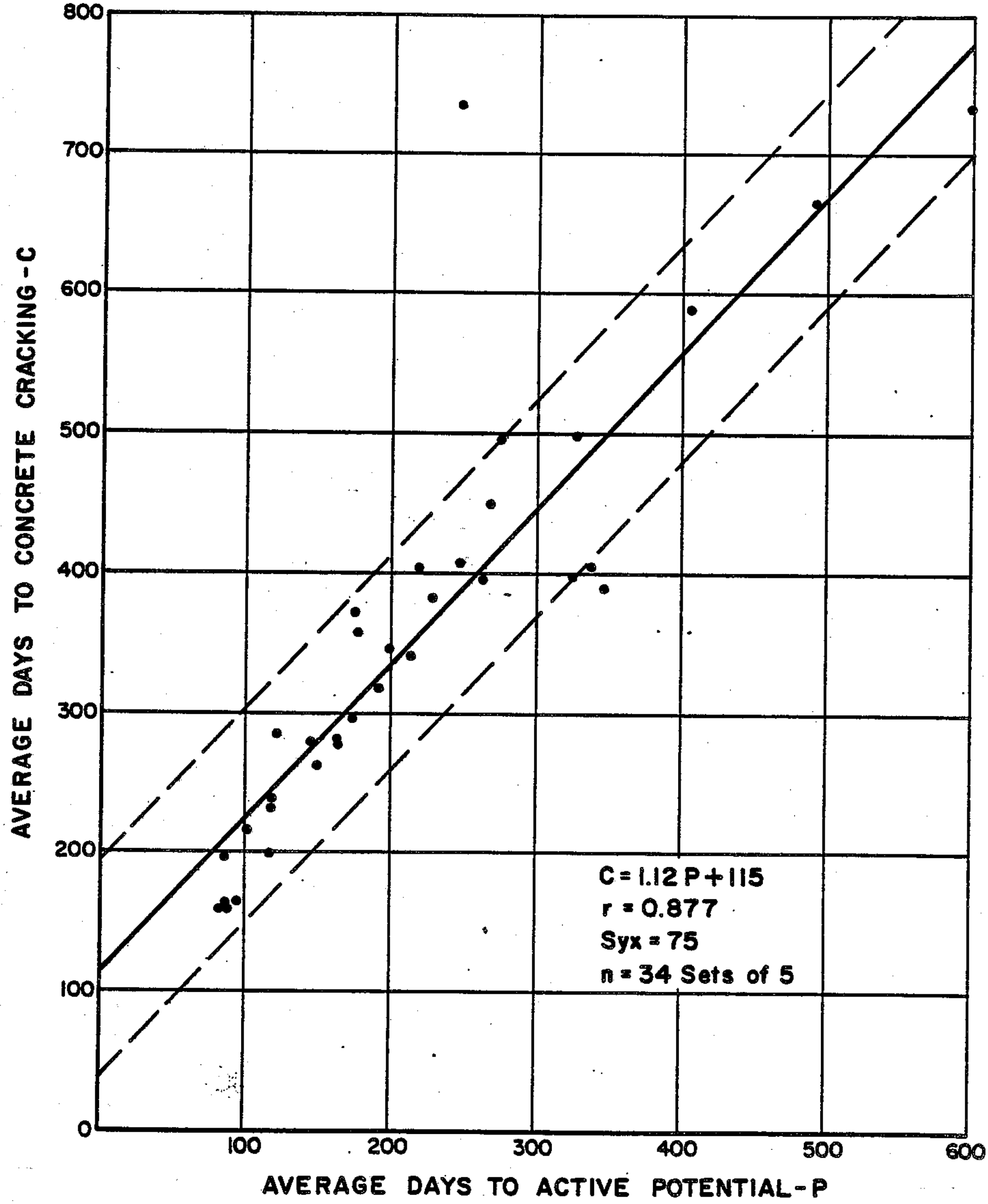
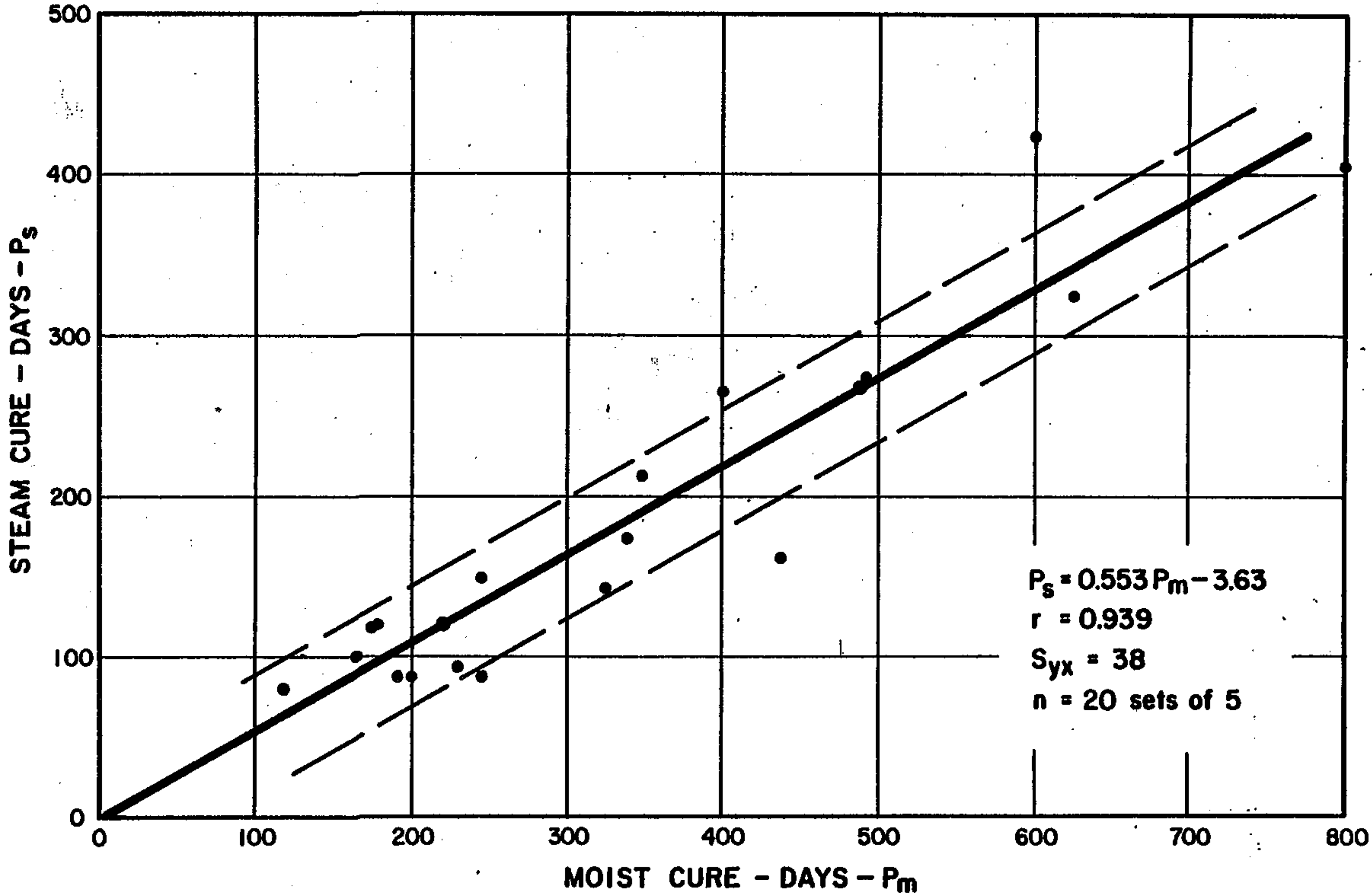


Figure 4

AVERAGE DAYS TO ACTIVE POTENTIAL OF STEAM VERSUS MOIST CURING



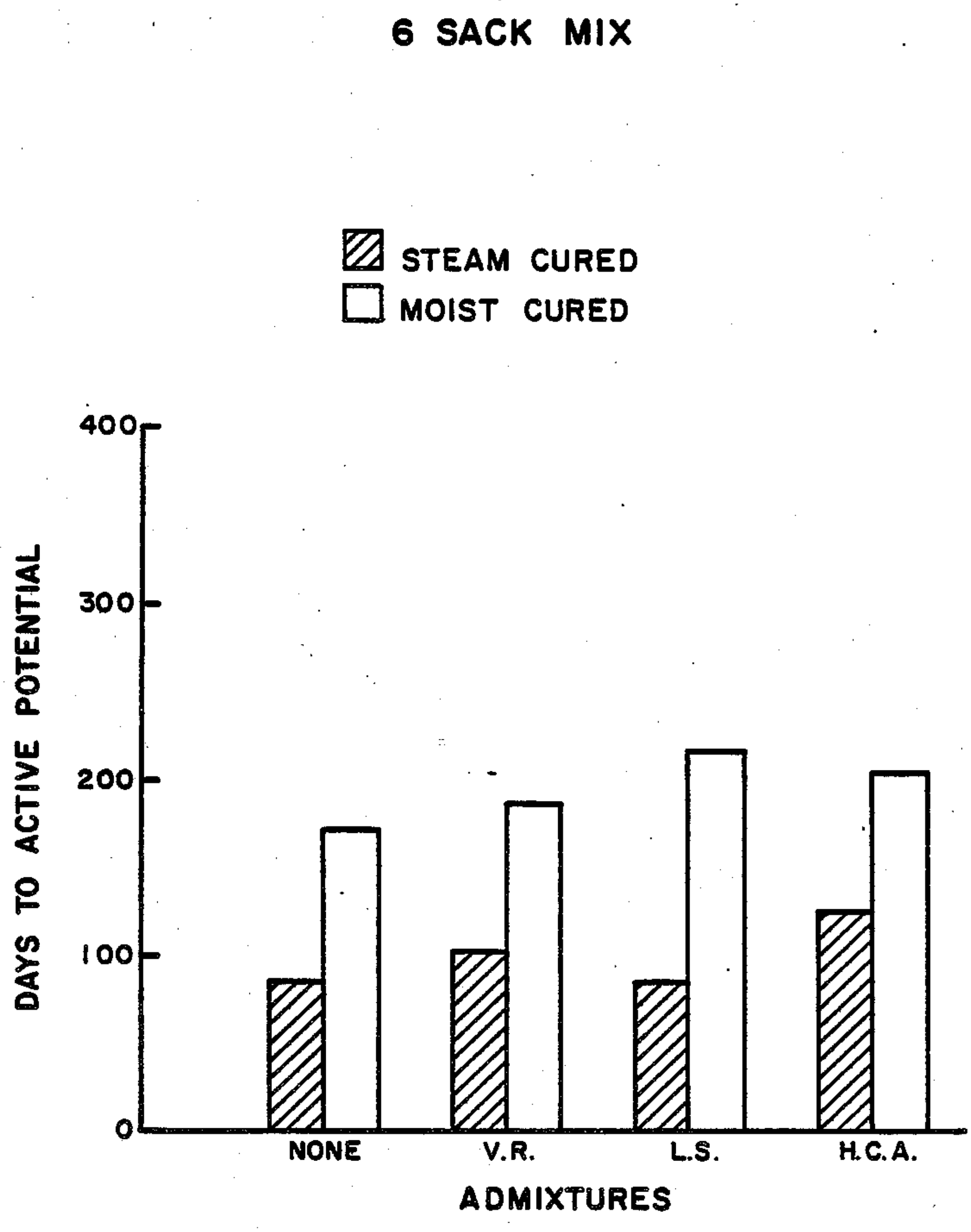

6 SACK MIX
STEAM CURED
MOIST CURED
DAYS TO ACTIVE POTENTIAL
400
300
200
100
0
NONE
V.R.
L.S.
H.C.A.
ADMIXTURES

Figure 6

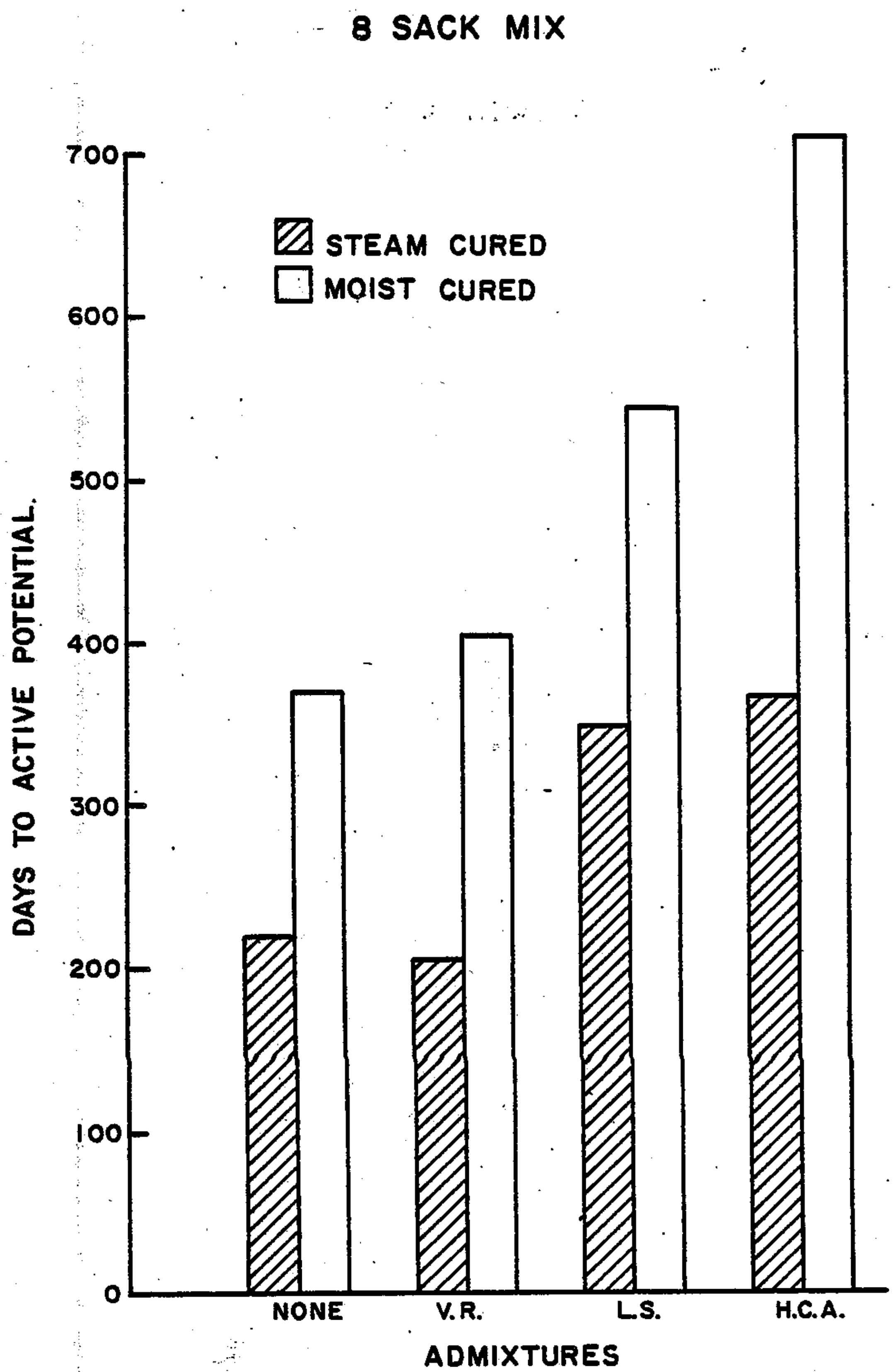

8 SACK MIX
STEAM CURED
MOIST CURED
DAYS TO ACTIVE POTENTIAL
700
600
500
400
300
200
100
0
NONE
V.R.
L.S.
H.C.A.
ADMIXTURES

Figure 7

DISTRIBUTION OF METAL LOSS

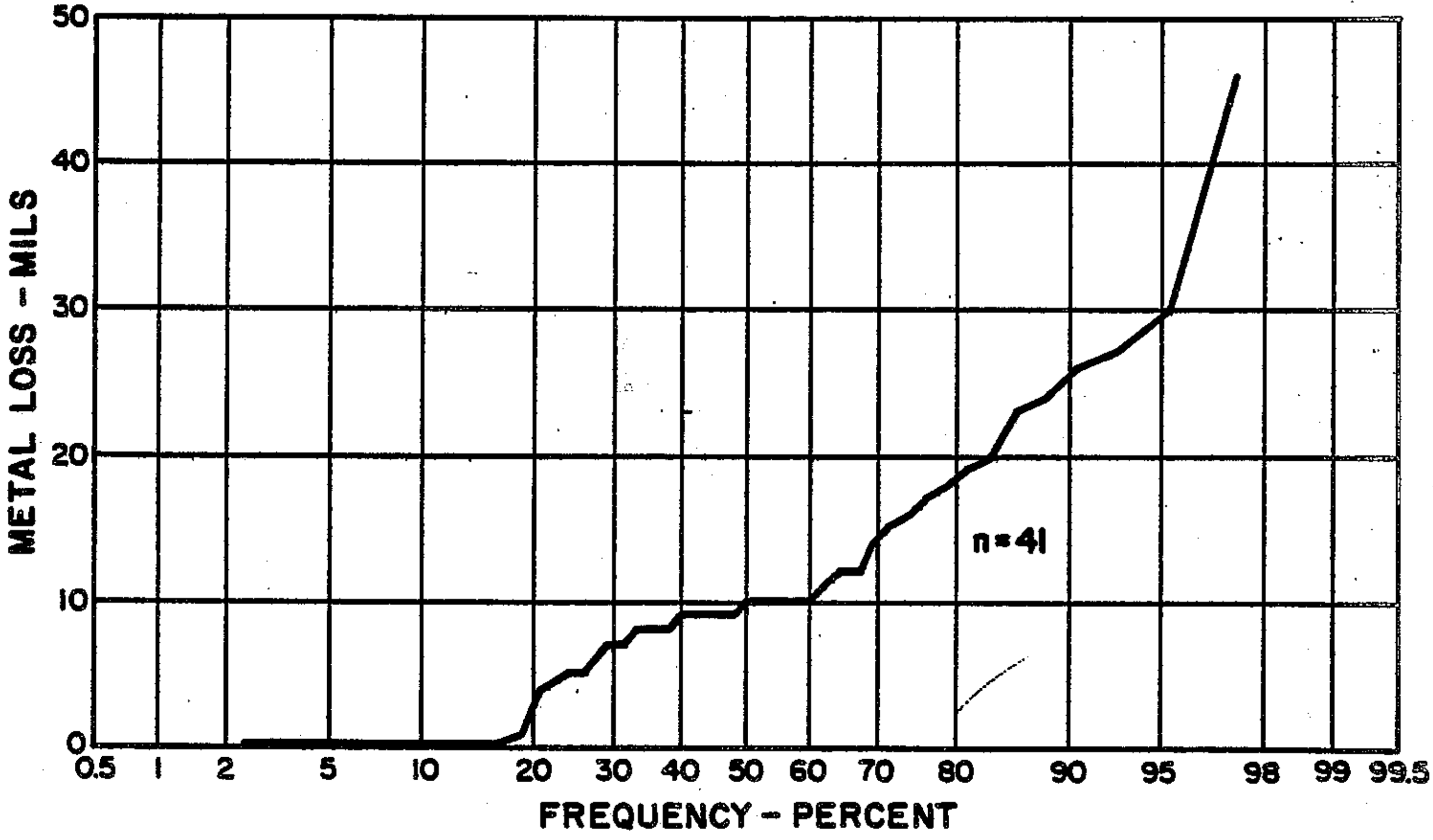
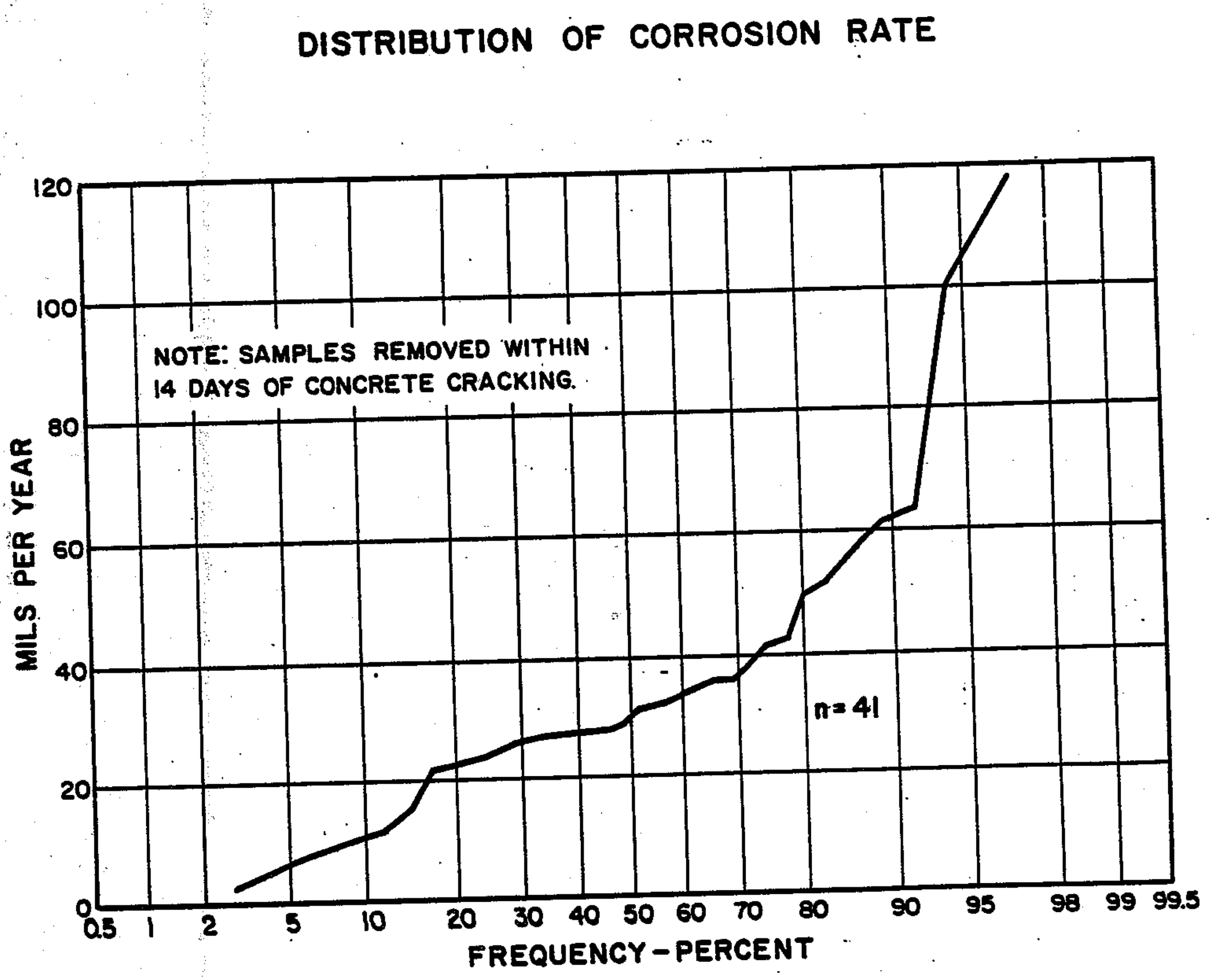


Figure 8



DISTRIBUTION OF CORROSION RATE
NOTE: SAMPLES REMOVED WITHIN 14 DAYS OF CONCRETE CRACKING.
n = 41
MILS PER YEAR
120
100
80
60
40
20
0
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.5
FREQUENCY - PERCENT

DISTRIBUTION OF CHLORIDE IN CONCRETE FRAGMENTS

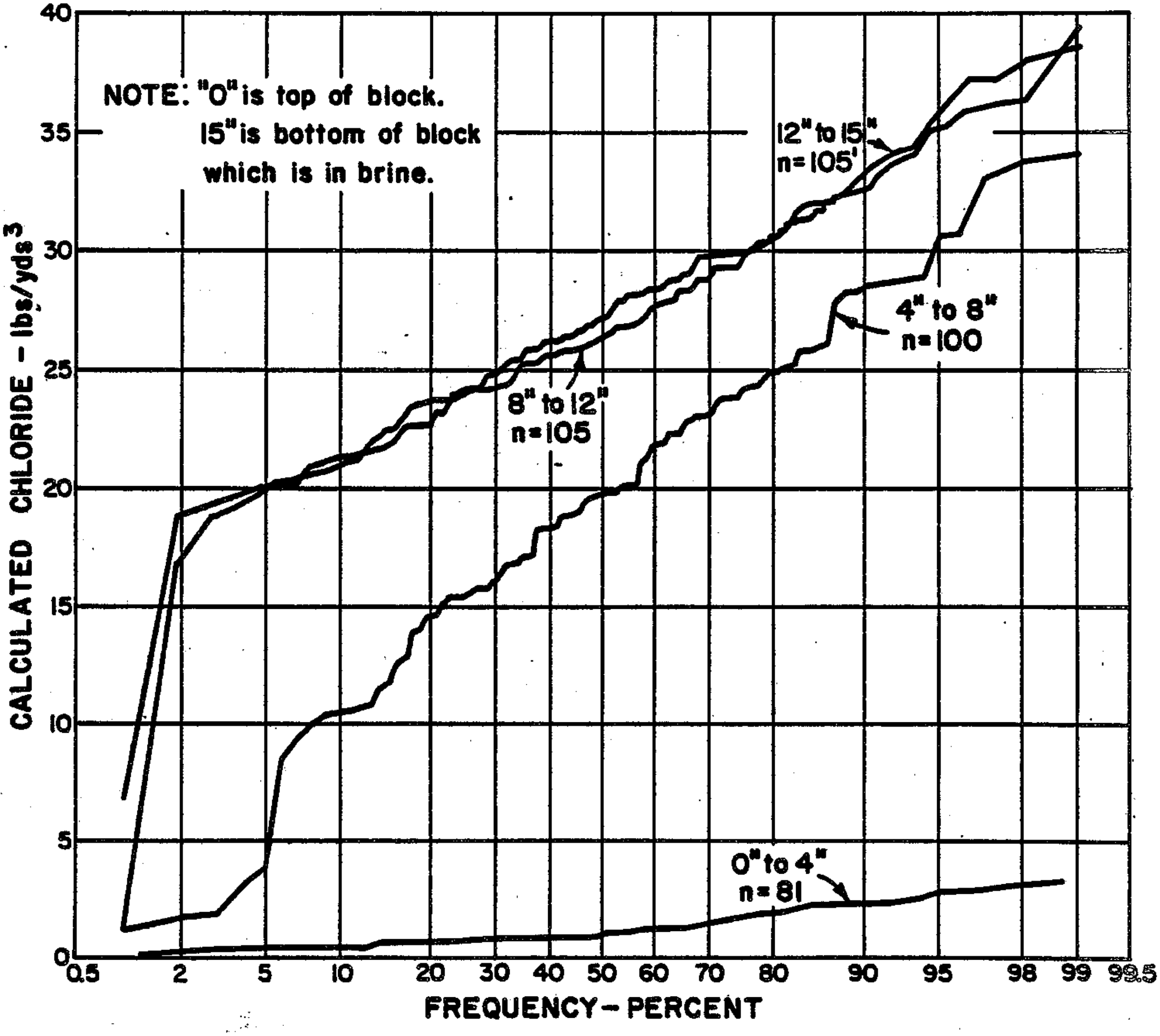
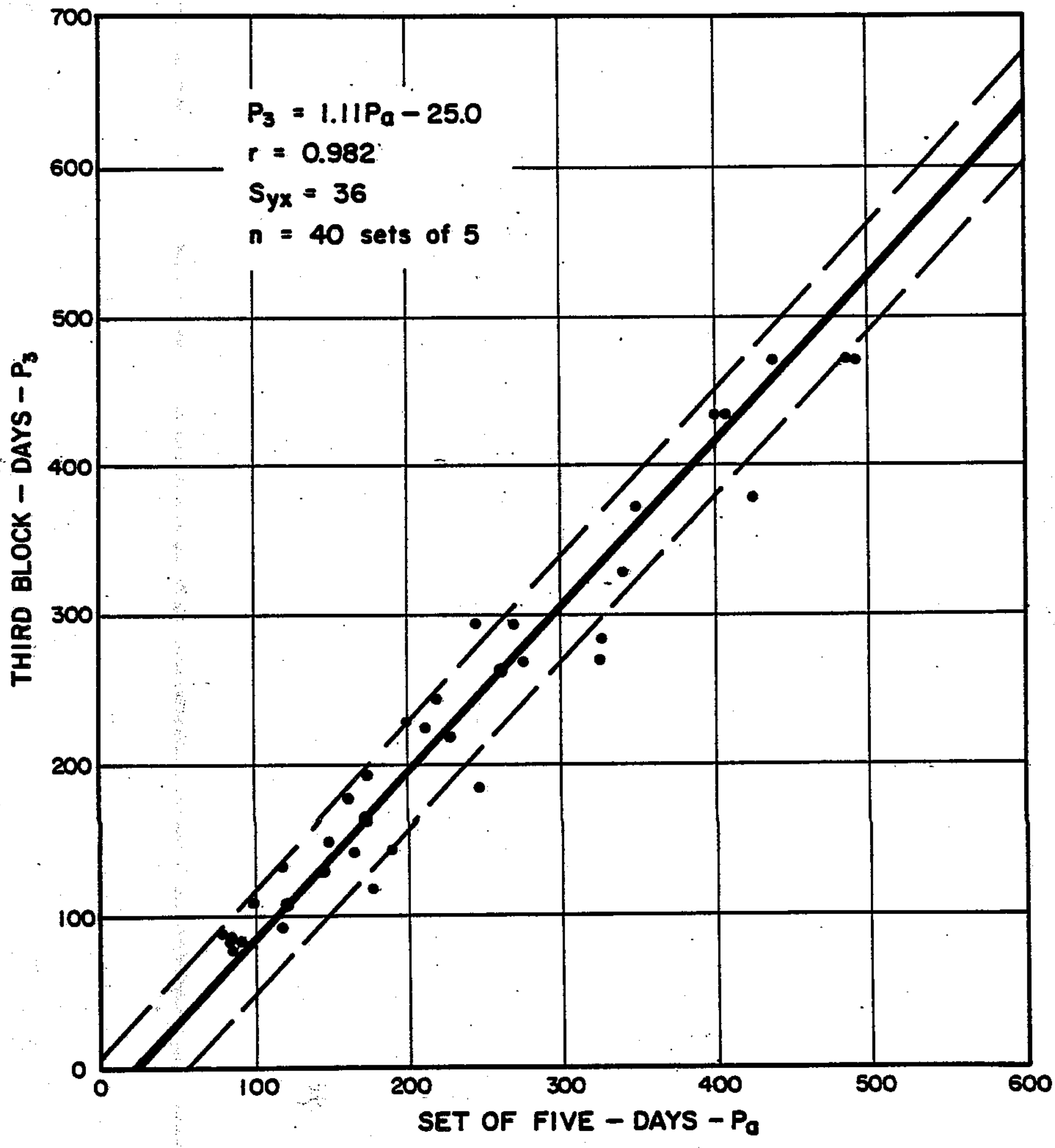


Figure 10

DAYS TO ACTIVE POTENTIAL OF THIRD BLOCK VERSUS SET OF FIVE



DAYS TO ACTIVE POTENTIAL OF N th BLOCK VERSUS SET OF FIVE

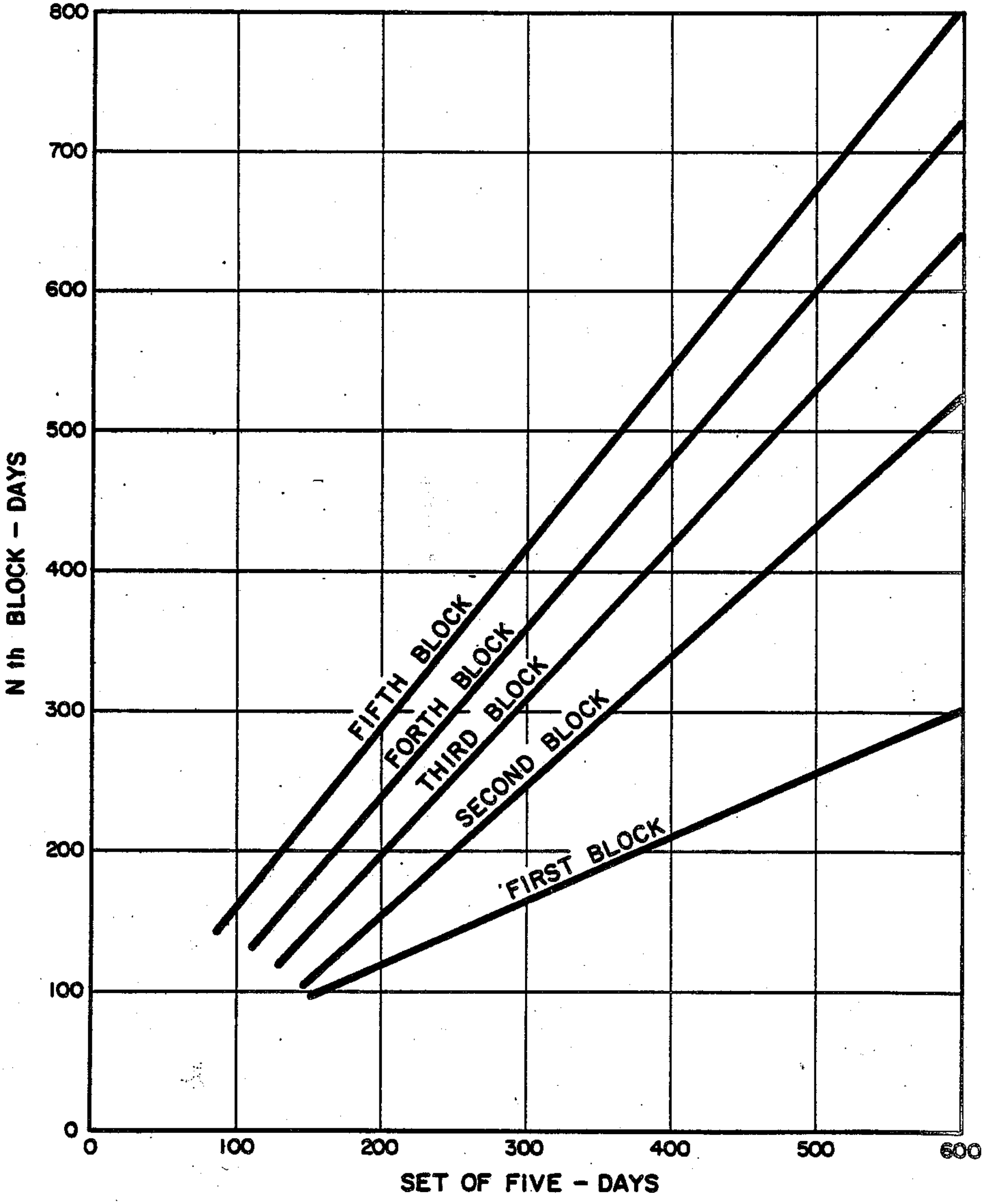
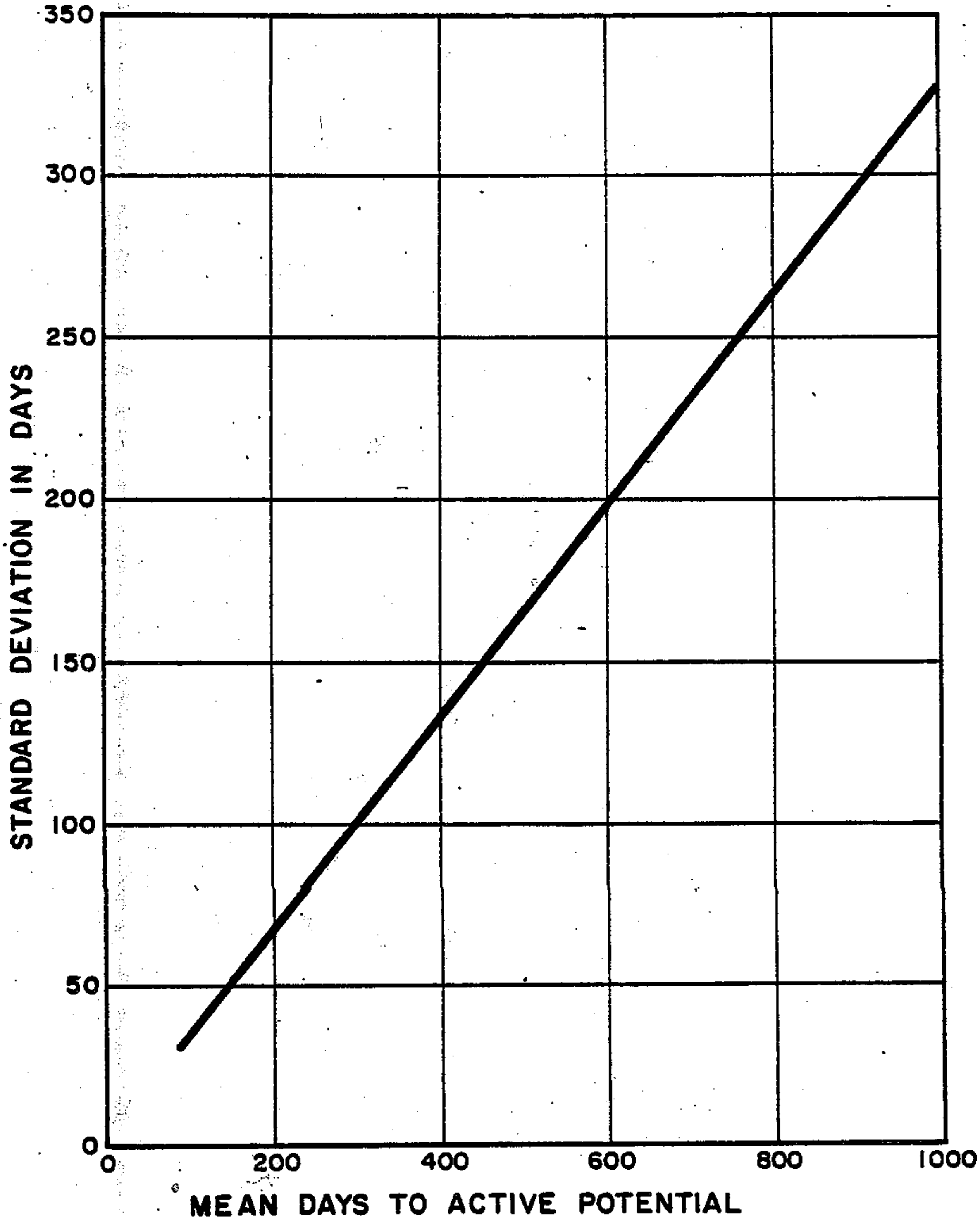


Figure 12

STANDARD DEVIATION
VERSUS ACTIVE POTENTIAL



COEFFICIENT OF VARIATION VERSUS ACTIVE POTENTIAL

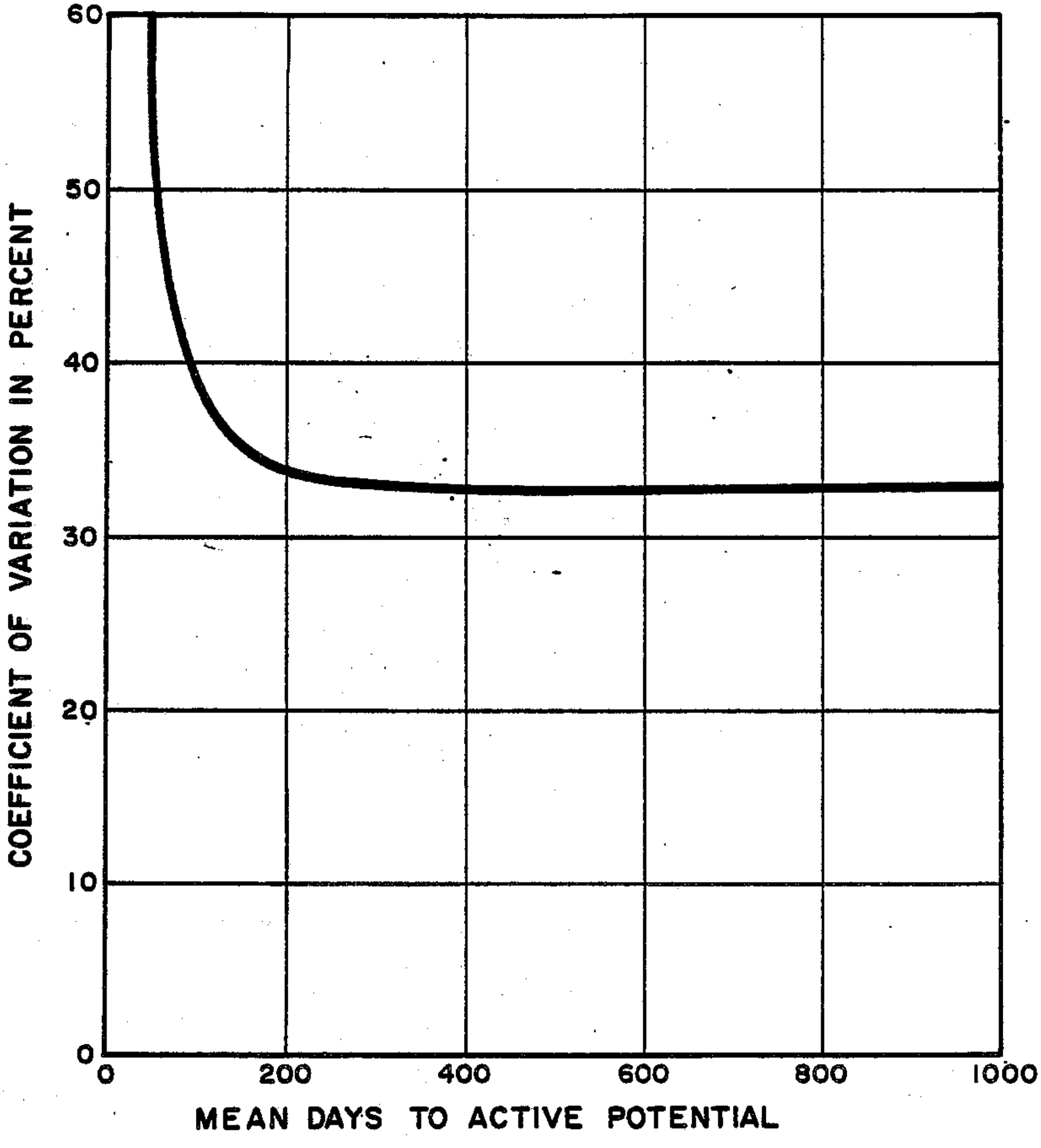
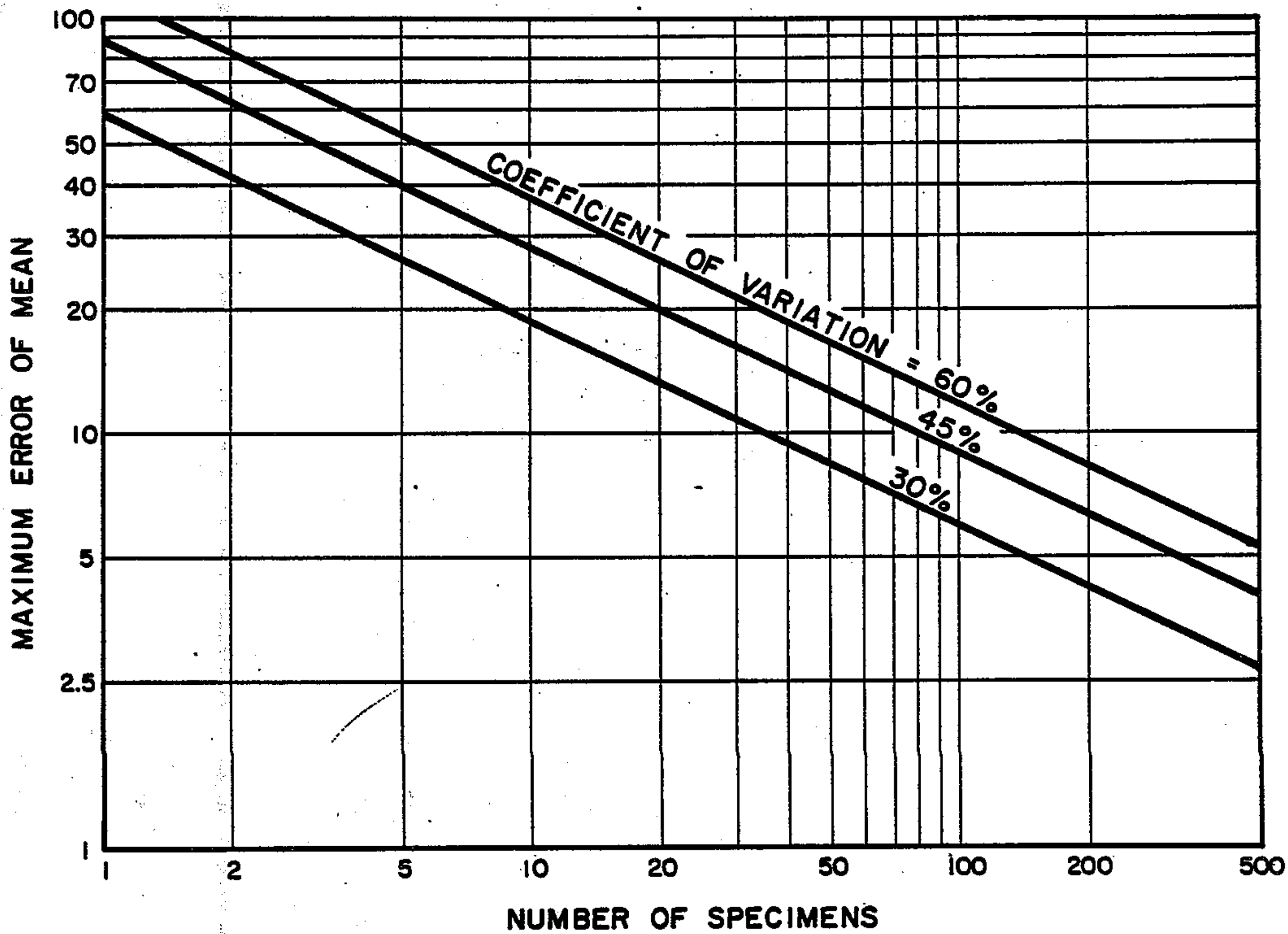


Figure 14

MAXIMUM ERROR VERSUS NUMBER OF SPECIMENS*



* 95% Confidence Level

HIGHWAY RESEARCH REPORT

POSEY CANYON ARCH CULVERT INSTRUMENTATION

68-16

STATE OF CALIFORNIA

TRANSPORTATION AGENCY

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

MATERIALS AND RESEARCH DEPARTMENT

RESEARCH REPORT

NO. M & R 36304

Prepared in Cooperation with the U.S. Department of Transportation, Bureau of Public Roads April, 1968

STATE OF CALIFORNIA—TRANSPORTATION AGENCY

RONALD REAGAN, Governor

DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT
5900 FOLSOM BLVD., SACRAMENTO 95819

April 1968

Lab. Auth. 36304

Mr. J. E. McMahon
Asst. State Highway Engineer, Bridges
California Division of Highways
Sacramento, California

Attention: Mr. G. D. Mancarti

Dear Sir:

Submitted herewith is a research report titled:

POSEY CANYON ARCH CULVERT INSTRUMENTATION
(A research project of the Bridge Department
California Division of Highways)

TRAVIS SMITH
Principal Investigator

W. G. WEBER, Jr.
Co-Investigator

Assisted By
L. R. Leech
E. C. Wrye

Very truly yours,

JOHN L. BEATON
Materials and Research Engineer

REFERENCE: Smith, Travis and Weber, W. G., Jr., "Posey Canyon Arch Culvert Instrumentation," State of California, Department of Public Works, Division of Highways, Materials and Research Department. Research Report 36304, April 1968.

ABSTRACT: The instrumentation of an 8-foot diameter reinforced concrete arch culvert under an embankment 240 feet in height is reported. Instruments were installed to provide for measurement of: pressure at the culvert-soil interface, soil pressure within the embankment, deformation of the culvert cross-section, and rotational movement of the culvert footing. Data from periodic readings of the instrumentation are presented. These data are not analyzed in this report.

KEY WORDS: Instrumentation, culverts, soil pressures, soil pressure/distribution, deformation, measurements, concrete arch culverts.

TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

I. Introduction

The high standards of a modern freeway system have presented an ever increasing number of demands and problems for the engineering profession. The mountainous terrain in California has required construction of a number of embankments in excess of 200 feet in height in recent years, and even higher fills are being built or are in the planning stage.

The performance of buried structures for conduction of water under high embankments was of immediate concern to the Bridge Department of the California Division of Highways. To gain additional information, the Department began a multi-faceted approach in gathering experimental data to help determine whether modified theories and design formulas are necessary to successfully meet the new criteria. This report concerns the work performed by the Materials and Research Department in instrumenting a concrete arch culvert in Posey Canyon and in acquiring the various measurements requested by the Bridge Department. Analysis of the data will be made by the Bridge Department.

This research work was performed in cooperation with the U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads.

II-1. The Culvert

Posey Canyon arch culvert is located on Interstate Route 5 in Los Angeles County approximately ten miles south of Gorman, California. Construction of the concrete culvert and the embankment were a portion of a contract to relocate and improve the existing roadway.

The 8-foot reinforced concrete culvert is 1100 feet long, and it is situated in the drainage channel at the bottom of Posey Canyon. An embankment 240 feet in height has been constructed over the culvert. (See Figures 5 and 6). Plan and cross-section layouts with instrumented test stations indicated are attached to this report, Figures 1 and 44 to 52.

Field instrumentation installations began with the second section of concrete arch which was poured on May 21, 1966. A "zero" set of readings for the instrumentation was taken at the completion of the culvert backfill operations during the week of July 25, 1966. Additional readings were taken at designated intervals throughout construction of the embankment and readings are being continued at designated time intervals since completion of the fill.

II-2. Instrumentation Summary

The instrumentation consisted of the following:

Information Desired	Measurement Method	Test Stations	Figures
Pressure at the Culvert-Soil Interface	Standard Carlson Soil Stress Meters	1 to 7, incl.	7 to 11, incl.
Pressure within the embankment soil	Modified Carlson Soil Stress Meters- (See Text)	6 to 9, incl. and 3	16
Deformation in Culvert Cross-Section	Extensometer measurements between steel balls attached to culvert walls	All	14
Rotational Movement of the Culvert Footing	Angle calculated from data obtained by using precision spirit level and inside micrometer over two points on the left and right footings	1 to 7, incl. and one location between Stas. 8 & 9	15

II-3. Soil Pressure Measurements

Nine stress meters, manufactured by Roy Carlson of Berkeley, California, were installed in the periphery of the arch culvert at each of seven test stations. Previous experience with the Carlson stress meters had indicated that they were very reliable instruments. Although over a year has elapsed since the completion of the embankment, only three of the 63 meters installed have ceased to function properly.

Soil pressures within the embankment are being measured by stress meters located on a horizontal plane six feet above the crown at 5 of the test sections. These stress meters were fabricated by the Materials and Research Department using Carlson pressure cell blanks and LVDT transducers. Information on their fabrication and the associated readout device is detailed in Section VII of "Instrumentation for the Apple Canyon Culvert," Materials and Research Department Project Report 36360 dated December, 1966. For protection, the stress meter cables were encased in flexible metal electrical conduit extending from the meter to inside the culvert where the readings are taken. Although it was intended to place three stress meters in the embankment at 5 of the test stations, only two meters were placed at test stations 6 and 8 because of construction activities at these locations at the time of installation. To date, it appears that all stress meters placed in the embankment are functioning properly.

The locations and readings obtained on the stress meters are shown in Figures 18 to 43.

II-4. Measurements of Culvert Deformation

To facilitate measurements for variations in the culvert span, steel balls were embedded in the walls on the left and right sides at each test station. These balls are located about one foot above the spring lines. An extensometer equipped with an extension rod to provide the necessary length is used to make readings periodically. (Figure 14). The accuracy of the measurements is in the order of $\pm$ 0.003 inch.

A tabulation of the readings obtained thus far is shown in Table 1.

II-5. Measurements of Culvert Footing Rotation

Two points were set on both the left and right footings at each test station for the purpose of measuring rotational movement. A tripod device equipped with leveling screws is positioned over the two points to support a precision spirit level. An inside micrometer is then used to measure from the lower point on the footing to the base of the level. (See Figure 15).

We believe the accuracy of the measurements, used to calculate the data shown in Tables 2 and 3 to be in the order of $\pm$ 0.002 inch.

III. Resumé of Data Acquisition Points

	Number of data points
Soil pressures both in the arch and in the embankment:	76
Temperature determination in the arch:	63
Variations in culvert span:	9
Culvert footing measurements for rotational movement:	16
Total number of data points	164

The most recent set of readings to be taken at Posey Canyon was completed on January 17, 1968.

Readings for the eighteen and twenty-four month intervals after completion of the embankment remain to be taken.

IV. Field Problems with Scheduled Readings

Heavy winter rains during November, 1966, caused a large accumulation of silt and mud at the outlet of the culvert, and this material backed up into the barrel of the arch for a distance of about 150 feet. Test stations one and two were covered and were inaccessible for footing rotation and extensometer readings until December, 1967, at which time the contractor removed the mud and silt and made a general clean-up of the area.

The instrumentation at stations one and two was found to be in good working order after the clean-up by the Contractor.

A portion of the readings taken during the summer and fall of 1967 were found to be in error due to corrosion of the wire leads from the stress meters to the read-out instrument connections. As this condition became progressively worse with time, it was necessary to do some maintenance work to restore the instrumentation to usefulness. This was accomplished in December 1967, and January 1968.

Table No. 1

Measurements for variations in culvert span using extensometer
POSEY CANYON ARCH CULVERT

Reading Date	Test Sta. No.	1	2	4	5	6	7	8	9	3
	"O" reading: in inches	96.0538	94.9169	94.7337	93.9423	94.3560	93.4500	93.8206	93.8608	93.4148
8-11-66		96.0902	94.9300	94.7534	93.9423	94.3560	93.4094	93.8076	93.8087	93.3745
8-16-66		96.1030	94.9371	94.7541	93.9338	94.3356	93.4520	93.8342	93.8652	93.4290
8-25-66		96.1060	94.9449	-	93.9434	-	93.4619	-	-	93.4150
9-14-66		*96.1032	94.9338	94.7736	93.9526	94.3272	93.4760	93.8494	93.8615	93.4300
9-27-66		96.1230	94.9370	94.7691	93.9616	94.3254	93.4775	93.8505	93.8554	93.4249
10-19-66		96.1083	94.9355	94.7817	93.9694	94.3233	93.4780	93.8483	93.8535	93.4366
11-10-66		*	*	94.7764	93.9864	94.3267	93.4748	93.8499	93.8509	93.4404
12-14-66		*	94.9270	94.7773	93.9900	94.3162	93.4766	93.8462	93.8518	93.4363
1-10-67		*	94.9284	94.7733	93.9936	94.3109	93.4765	93.8442	93.8423	93.4317
2-7-67		*	94.9260	94.7705	93.9945	94.3074	93.4700	93.8395	93.8370	93.4315
2-28-67		*	94.9268	94.7692	93.9985	94.3041	93.4750	93.8410	93.8374	93.4310
5-22-67		*	-	94.7670	94.0128	94.2947	93.4778	93.8385	93.8324	93.4313
8-16-67		*	94.9275	94.7673	94.0104	94.2953	93.4988	93.8374	93.8297	93.4282
12-22-67		96.1160	94.9420	94.7790	94.0350	94.3150	93.4910	93.8470	93.8500	93.4450

*Covered with mud and debris.

Table No. 2

POSEY CANYON ARCH CULVERT

Culvert footing measurements for rotational movement (left footing)

Reading Date	Test Sta. No.	1	2	4	5	6	7	8-9	3
	Ref. Angle:	19°54'45"	11°14'27"	12°18'49"	12°28'53"	15°43'30"	13°22'21"	15°54'28"	6°06'36"
8-1-66		19 54 56	11 14 32	12 18 57					
8-9-66		19 51 24	11 12 07	12 12 00					
8-11-66		19 49 34	11 11 51		12 28 53			15 46 31	
8-18-66			11 12 47						
8-25-66				12 20 46					
8-26-66							13 20 27		
8-29-66					12 33 19				
9-1-66									5 58 21
9-14-66		20 07 29	11 07 16	12 07 01	12 23 31	15 45 27	13 22 27	15 46 31	6 02 04
9-27-66		19 48 25	11 07 51	12 07 07	12 23 23	15 46 09	13 18 10	15 45 25	6 00 36
10-20-66		19 45 40	11 08 28	12 12 46	12 22 26	15 50 07	13 17 17	15 52 02	6 00 33
11-9-66		*	*	12 01 39	12 19 20	15 42 25	13 16 23	15 42 47	5 55 05
12-13-66		-	-	12 12 14	12 20 45	15 46 27	13 19 47	15 47 14	5 59 31
1-12-67		-	-	12 10 49	12 20 49	15 46 40	13 19 55	15 44 53	5 46 44
3-1-67		-	-	12 06 42	12 20 45	15 46 56	13 17 53	15 46 08	5 59 26
5-22-67		-	-	12 04 13	12 15 05	15 46 26	13 19 12	15 44 51	5 59 11
8-19-67			11 01 15	12 04 09	12 19 18	15 42 27	13 14 37	15 39 32	5 58 47
12-22-67		19°54'04"	11°04'27"	12°03'35"	12°19'23"	15°42'06"	13°14'28"	15°39'43"	5°53'46"

*Points covered with mud and debris.

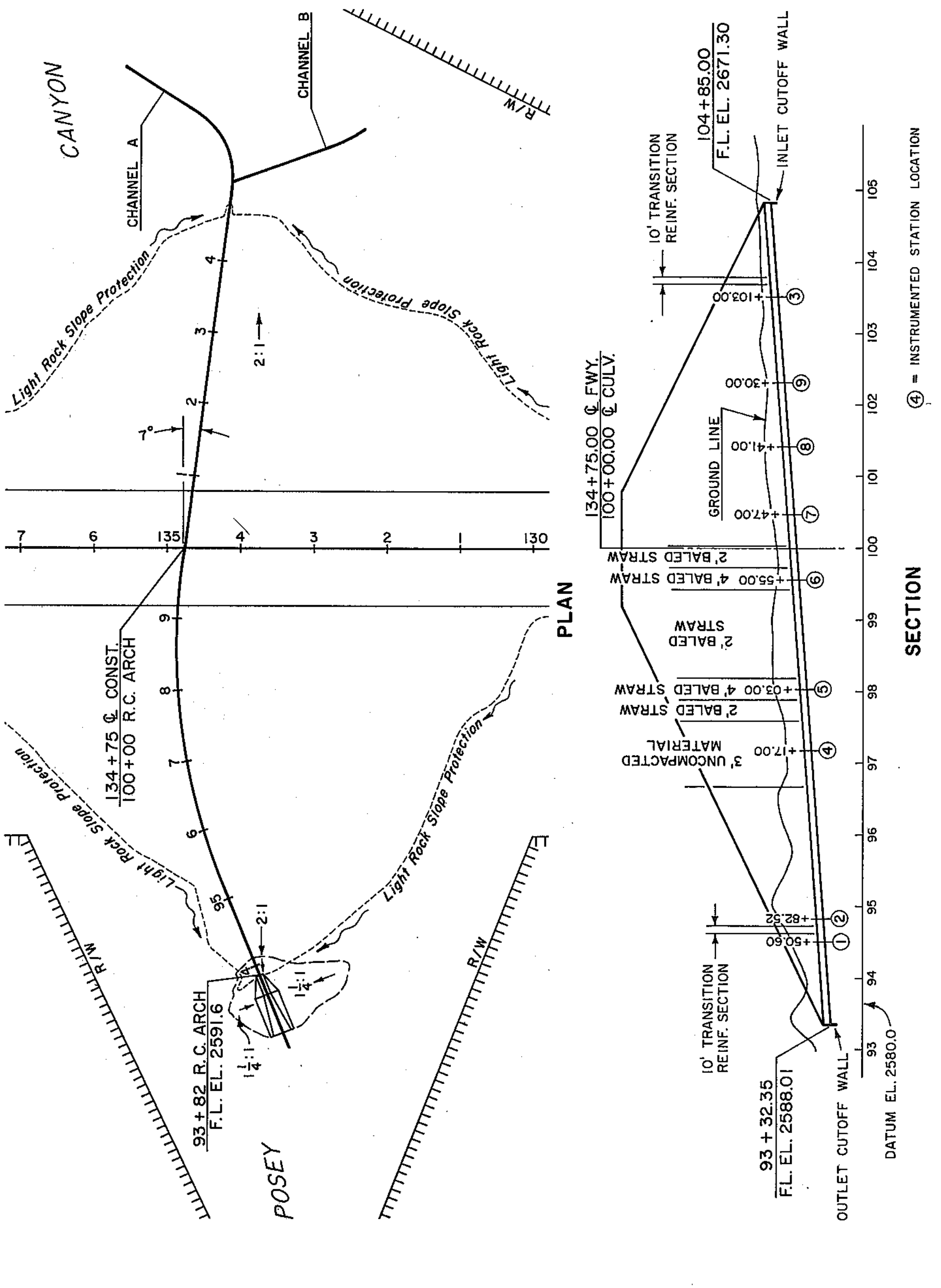
Table No. 3

POSEY CANYON ARCH CULVERT
Culvert footing measurements for rotational movement (right footing)

Reading Date	Test Sta. No.	1	2	4	5	6	7	8-9	3
	Ref. Angle:	20°41'21"	13°48'02"	10°50'37"	9°22'06"	13°03'09"	15°27'16"	10°08'01"	9°23'25"
8-1-66		20 41 29	13 48 02	10 50 39					
8-9-66		20 36 50	13 47 57	10 47 56					
8-11-66		20 38 10	13 44 45	10 55 23					9 23 25
8-18-66			13 44 24						
8-25-66									
8-26-66							15 27 12		
8-29-66					9 18 20				
9-1-66									9 20 53
9-14-66		20 37 59	13 40 31	10 42 01	9 15 43	13 00 22	15 29 56	9 58 07	9 17 55
9-27-66		20 36 42	13 40 27	10 40 47	9 16 29	13 04 36	15 26 15	9 54 59	9 13 15
10-20-66		20 33 41	13 42 48	10 42 16	9 16 02	12 55 41	15 25 41	9 55 52	9 12 25
11-9-66		*	*	10 39 32	9 18 58	12 54 09	15 24 50	9 56 11	9 13 04
12-13-66				10 40 05	9 12 48	12 58 57	15 31 25	9 56 47	9 22 41
1-12-67				10 39 50	9 13 02	12 41 56	15 29 27	9 55 31	9 13 18
3-1-67				10 40 16	9 18 20	12 57 19	15 35 27	9 54 43	9 20 40
5-22-67				10 37 05	9 11 45	12 57 17	15 27 55	9 52 51	9 09 40
8-19-67			13 40 22	10 36 48	9 11 01	12 52 29	14 40 36	9 52 02	9 18 28
12-22-67		20°37'19"	13°38'06"	10°38'06"	9°11'00"	12°53'02"	15°30'28"	9°50'52"	9°14'45"

*Points covered with mud and debris.

Figure 1


CANYON
CHANNEL A
CHANNEL B
R/W
Light Rock Slope Protection
Light Rock Slope Protection
Light Rock Slope Protection
Light Rock Slope Protection
2:1
7°
134+75 ℄ CONST.
100+00 R.C. ARCH
PLAN
93+82 R.C. ARCH
F.L. EL. 2591.6
1¼:1
1¼:1
2:1
R/W
R/W
POSEY
104+85.00
F.L. EL. 2671.30
INLET CUTOFF WALL
10' TRANSITION
REINF. SECTION
134+75.00 ℄ FWY.
100+00.00 ℄ CULV.
GROUND LINE
+103.00
+30.00
+41.00
+47.00
+55.00
+03.00
+17.00
+82.52
+50.60
2' BALED STRAW
4' BALED STRAW
2' BALED STRAW
4' BALED STRAW
2' BALED STRAW
3' UNCOMPACTED MATERIAL
10' TRANSITION
REINF. SECTION
93 + 32.35
F.L. EL. 2588.01
OUTLET CUTOFF WALL
DATUM EL. 2580.0
SECTION
④ = INSTRUMENTED STATION LOCATION

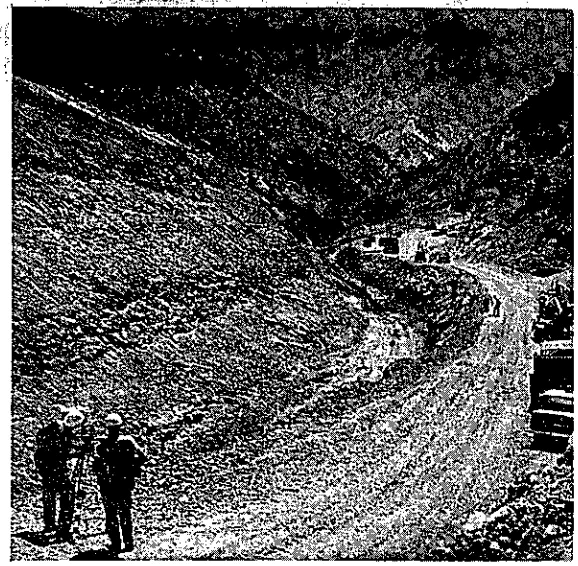


Figure 2 - Posey Canyon Channel excavation - Westward view.

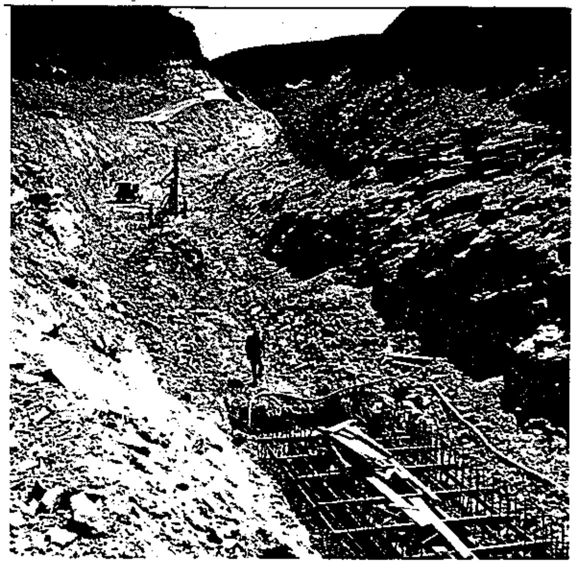


Figure 3 - Channel excavation Eastward view.

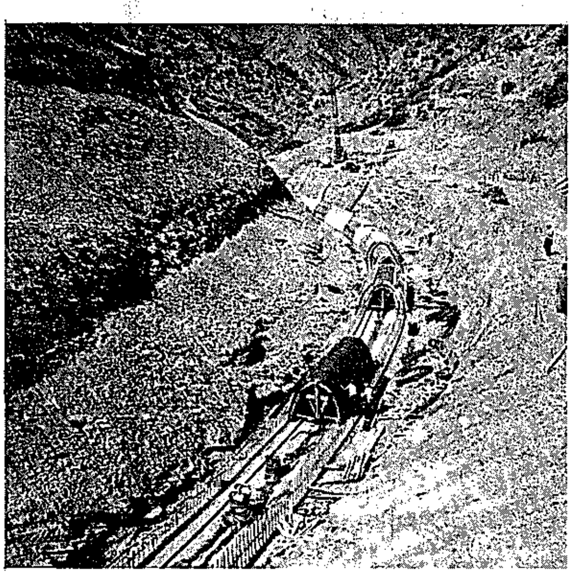


Figure 4 - Posey Canyon Arch Culvert during construction.

Figure 5 - Posey Canyon Fill during construction.

Figure 6 - Culvert inlet, photo taken from top of the embankment.

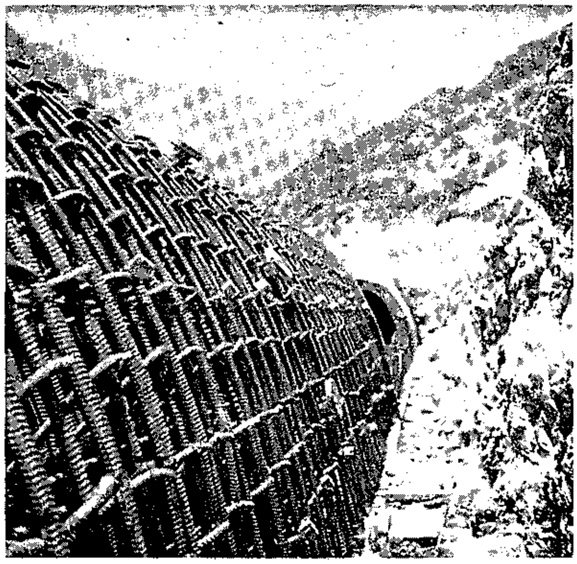


Figure 7 - Wooden blanks in place to form stress meter openings.

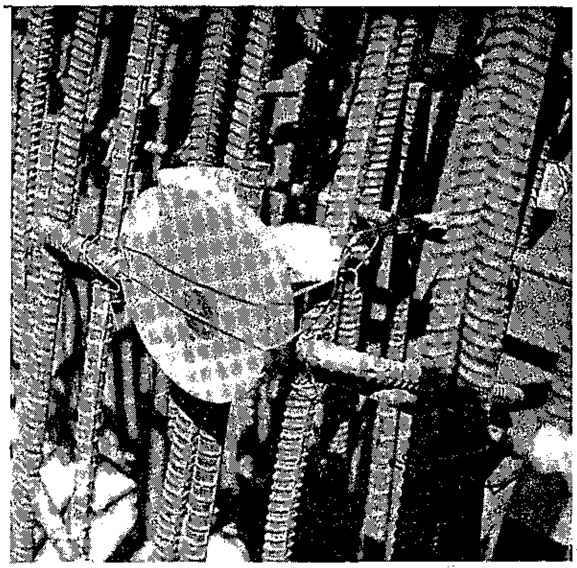


Figure 8 - Close-up view of wooden blank fastened to reinforcing steel.

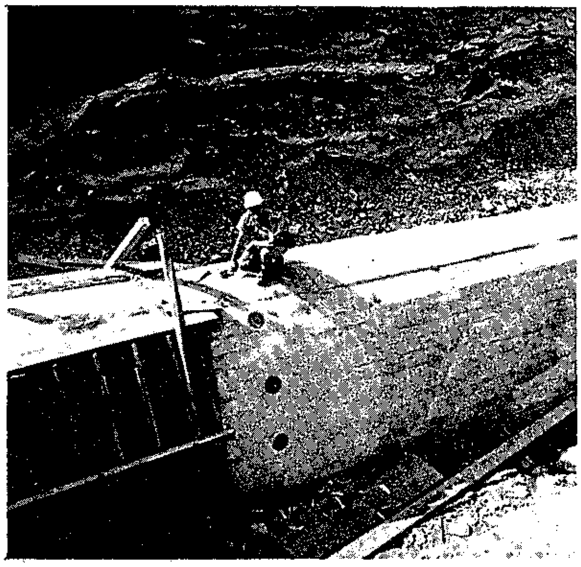


Figure 9 - Openings for stress meters at Station No. 2 after removal of wooden blanks.

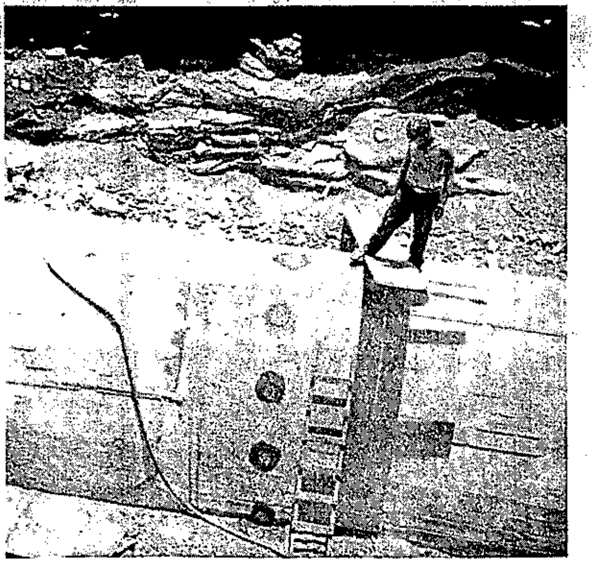


Figure 10 - Test Station No. 2 after stress meters were grouted in place.

Figure 11 - Close-up view of stress meters grouted in place.

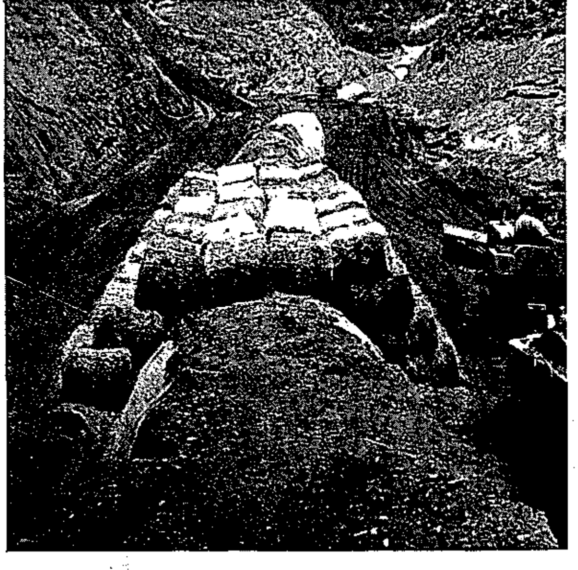


Figure 12 - Baled hay backfill Test Station No. 5.

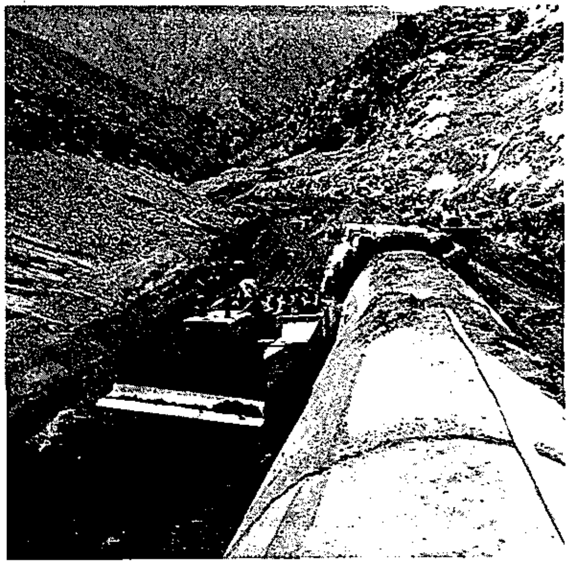


Figure 13 - Backfilling operations Posey Canyon Arch Culvert.

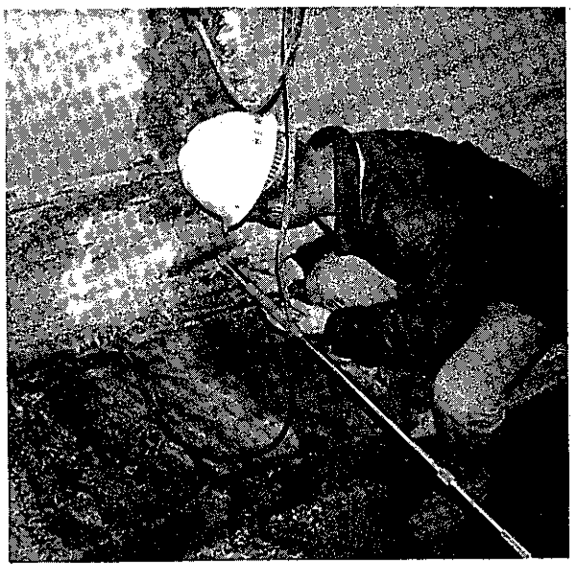


Figure 14 - Measurement with inside micrometer for variations in the culvert span.

Figure 15 - Precision level in place on culvert footing for measurement for rotational movement.

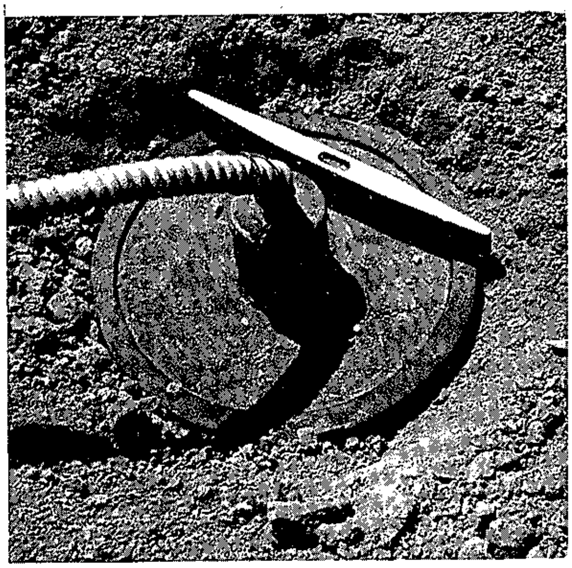


Figure 16 - Stress meter placed in the embankment to monitor earth pressure.

Figure 17 - Attaching readout instrument to stress meter cables inside of Posey Canyon Arch Culvert.

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5

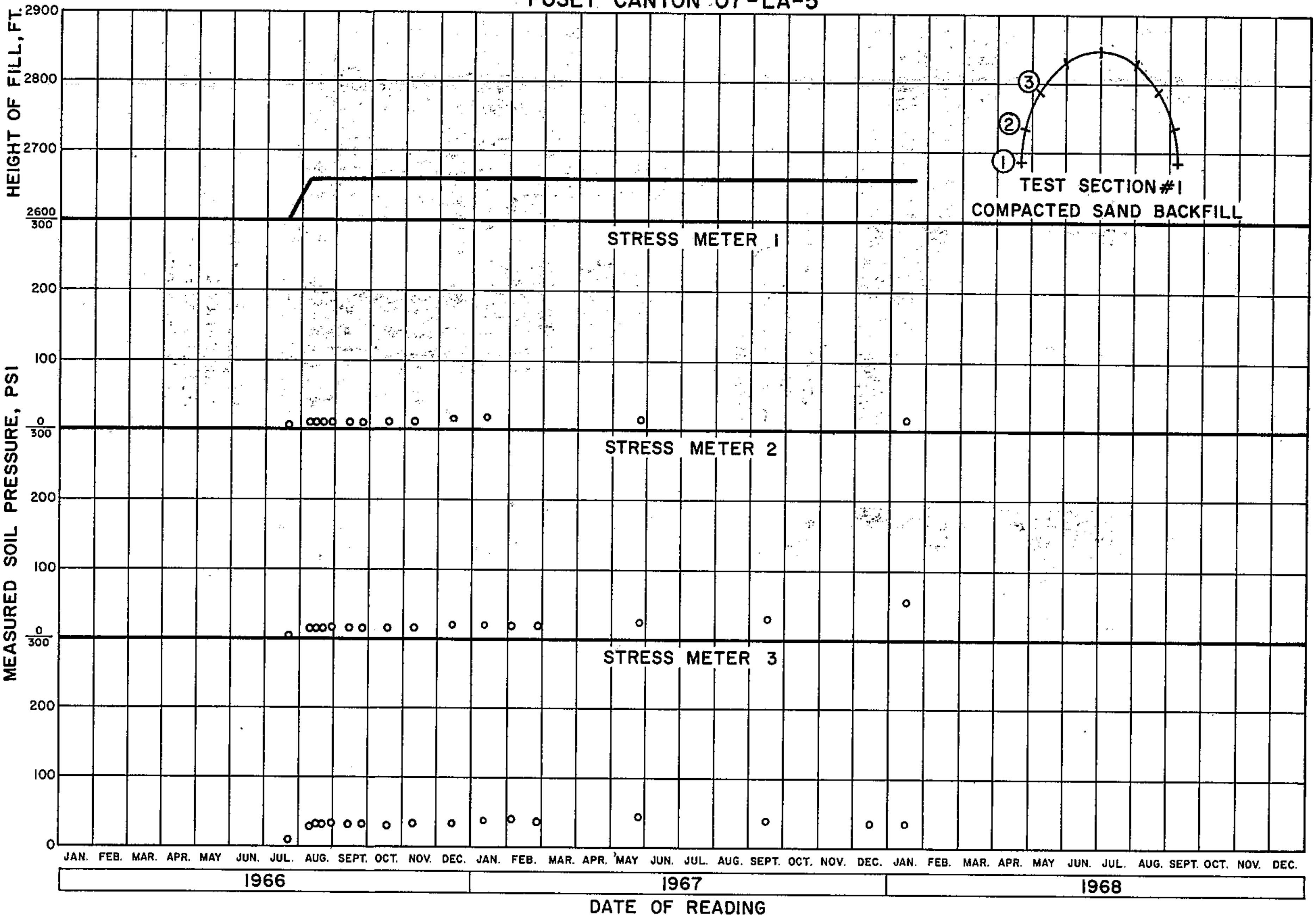


Figure 18

Figure 19

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5

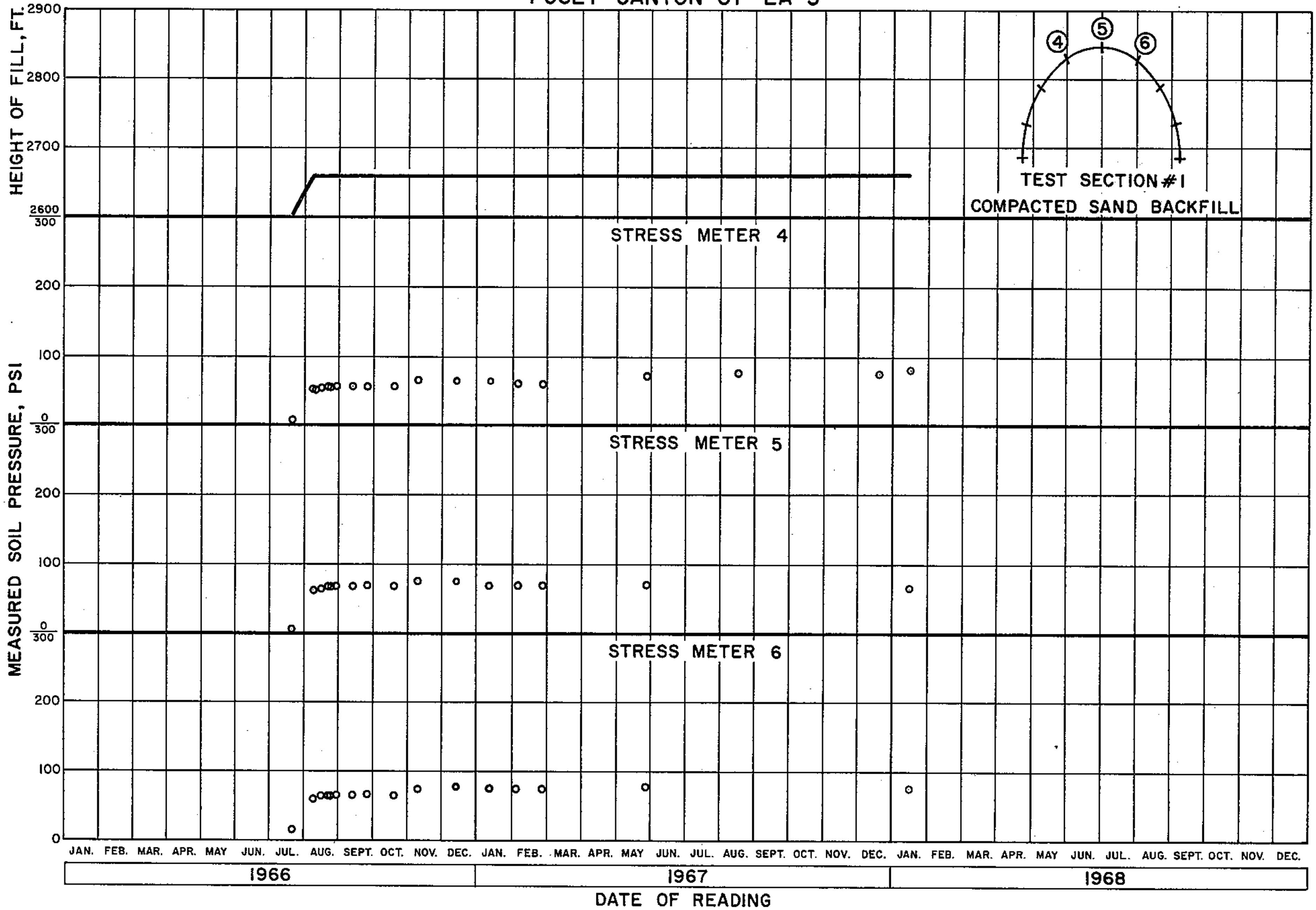
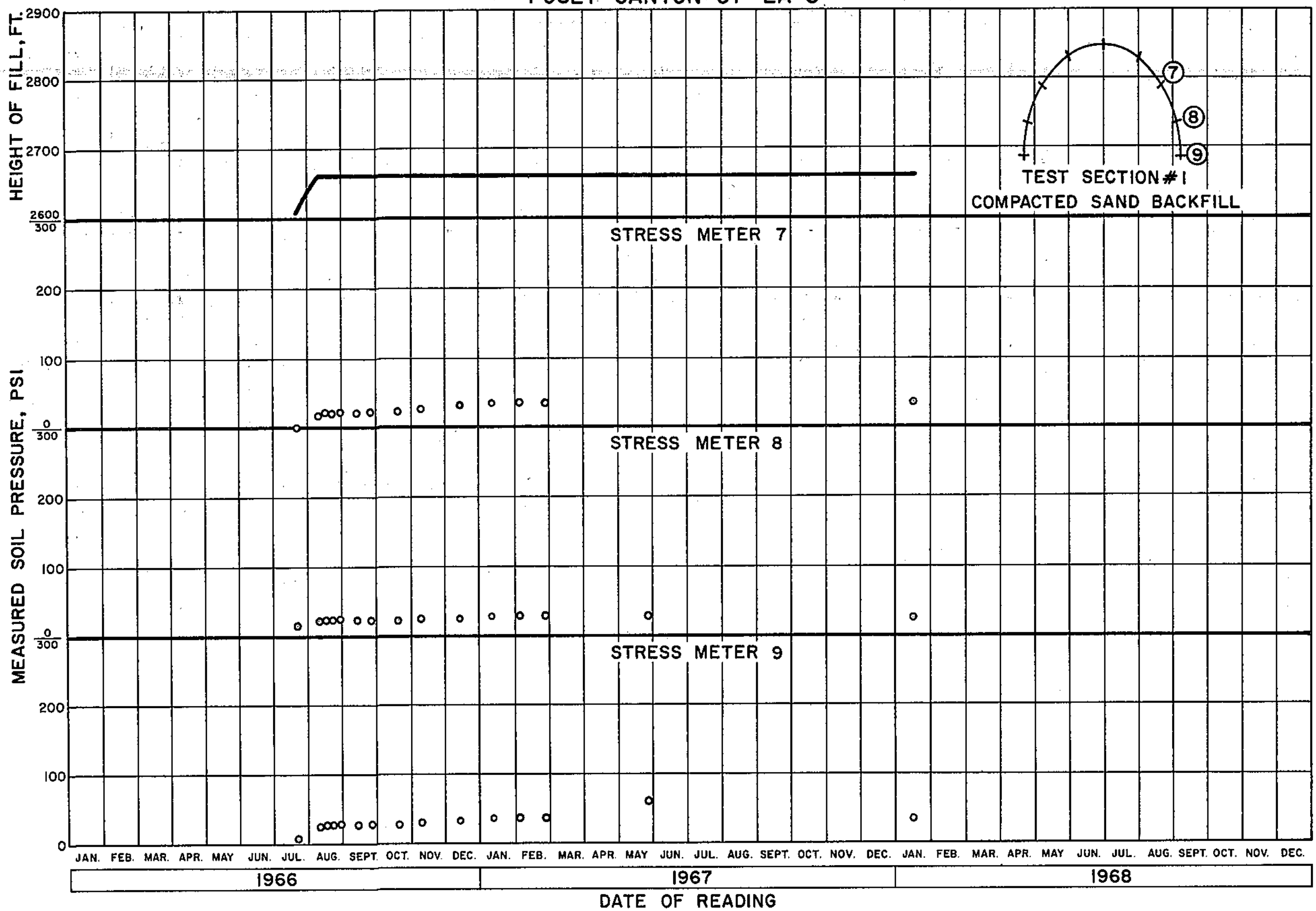


Figure 20

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5

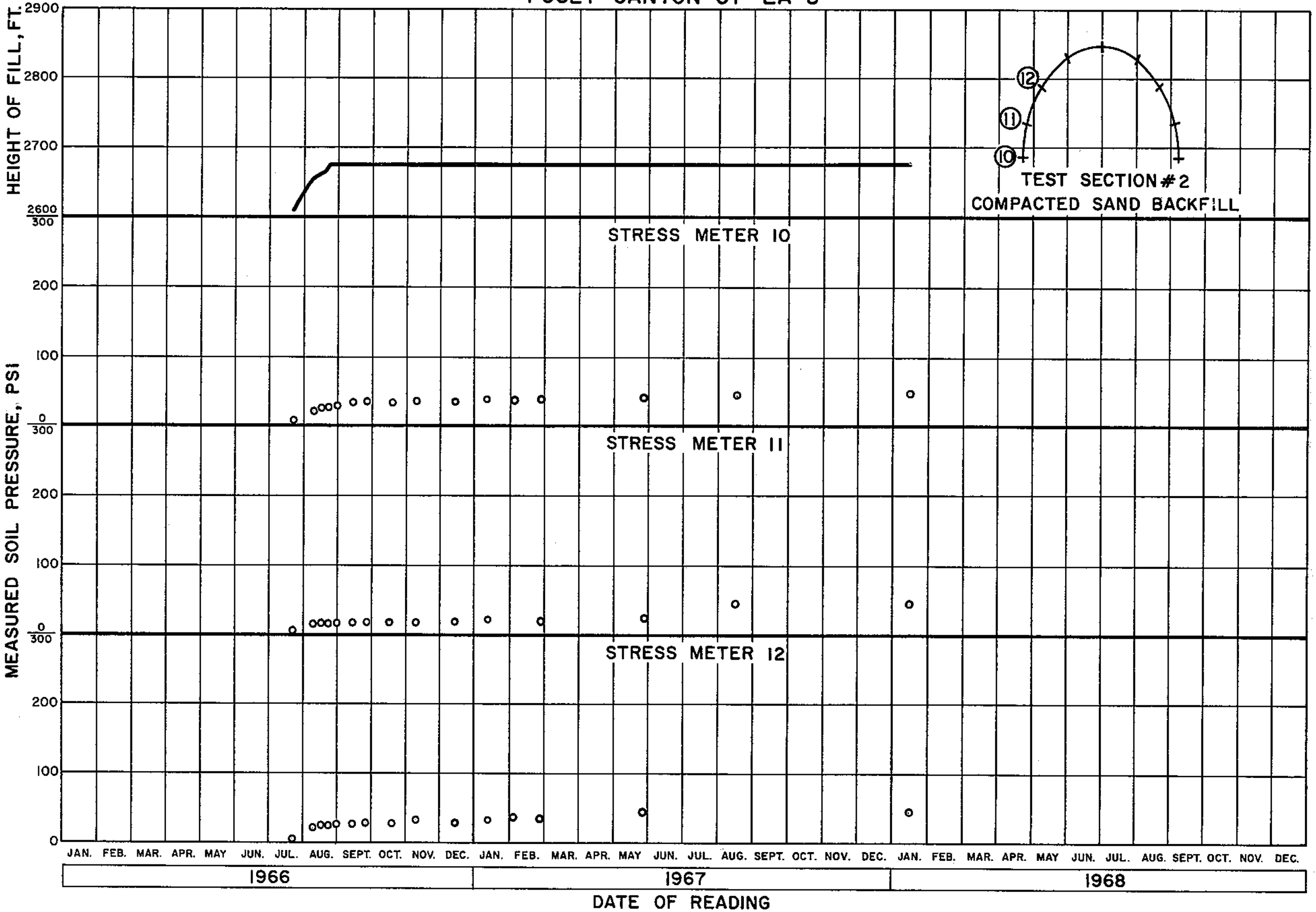
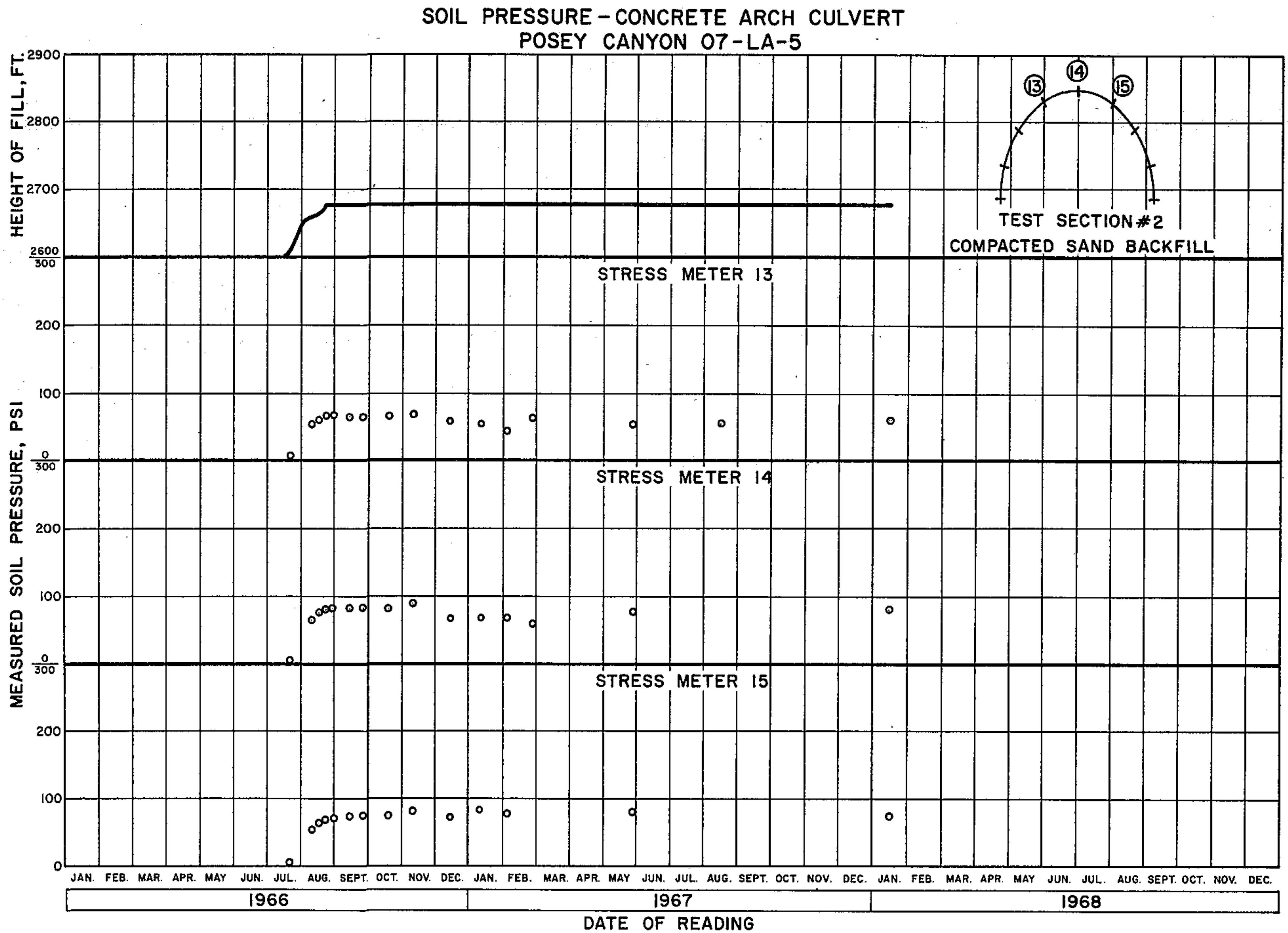


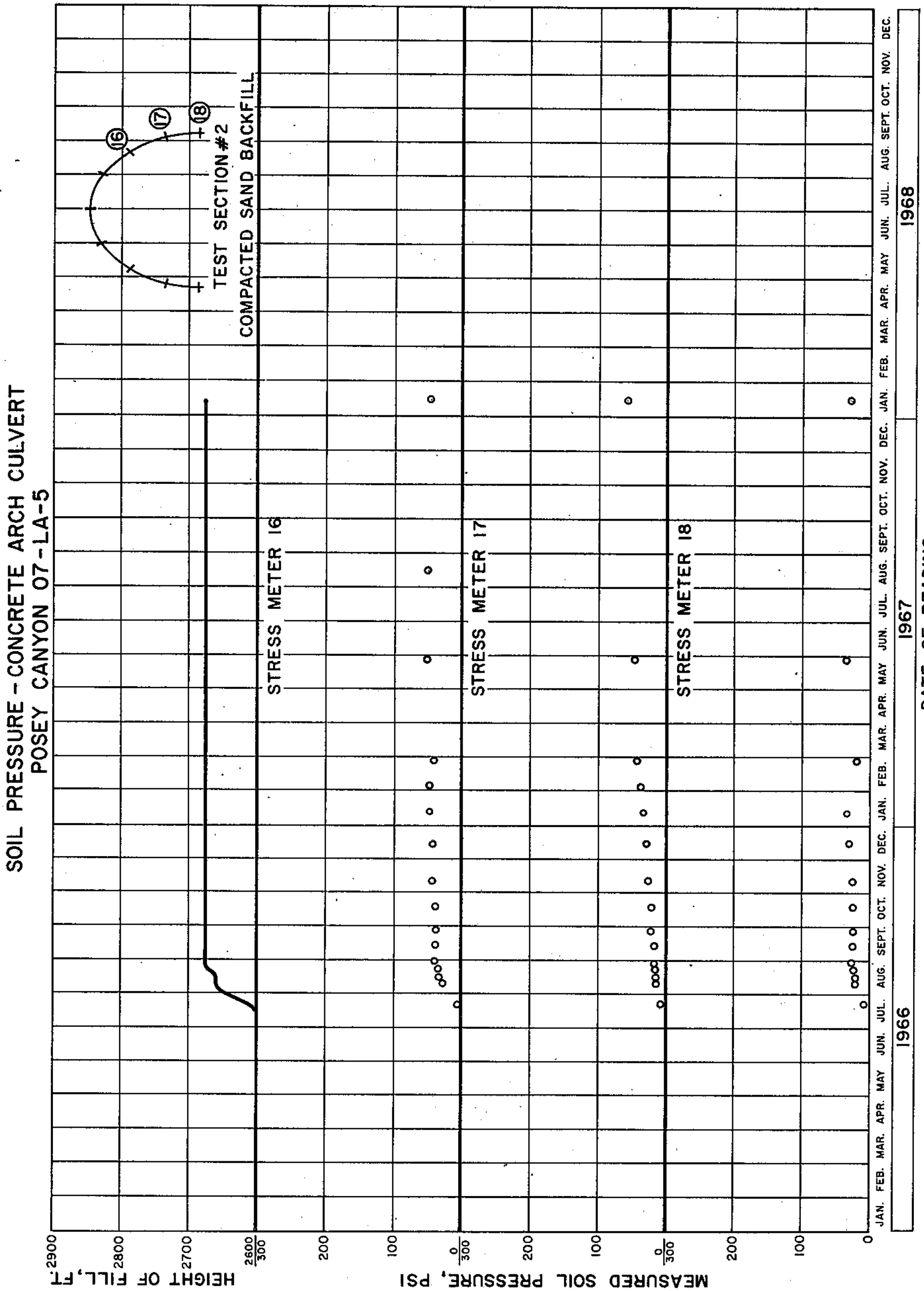
Figure 21

Figure 22



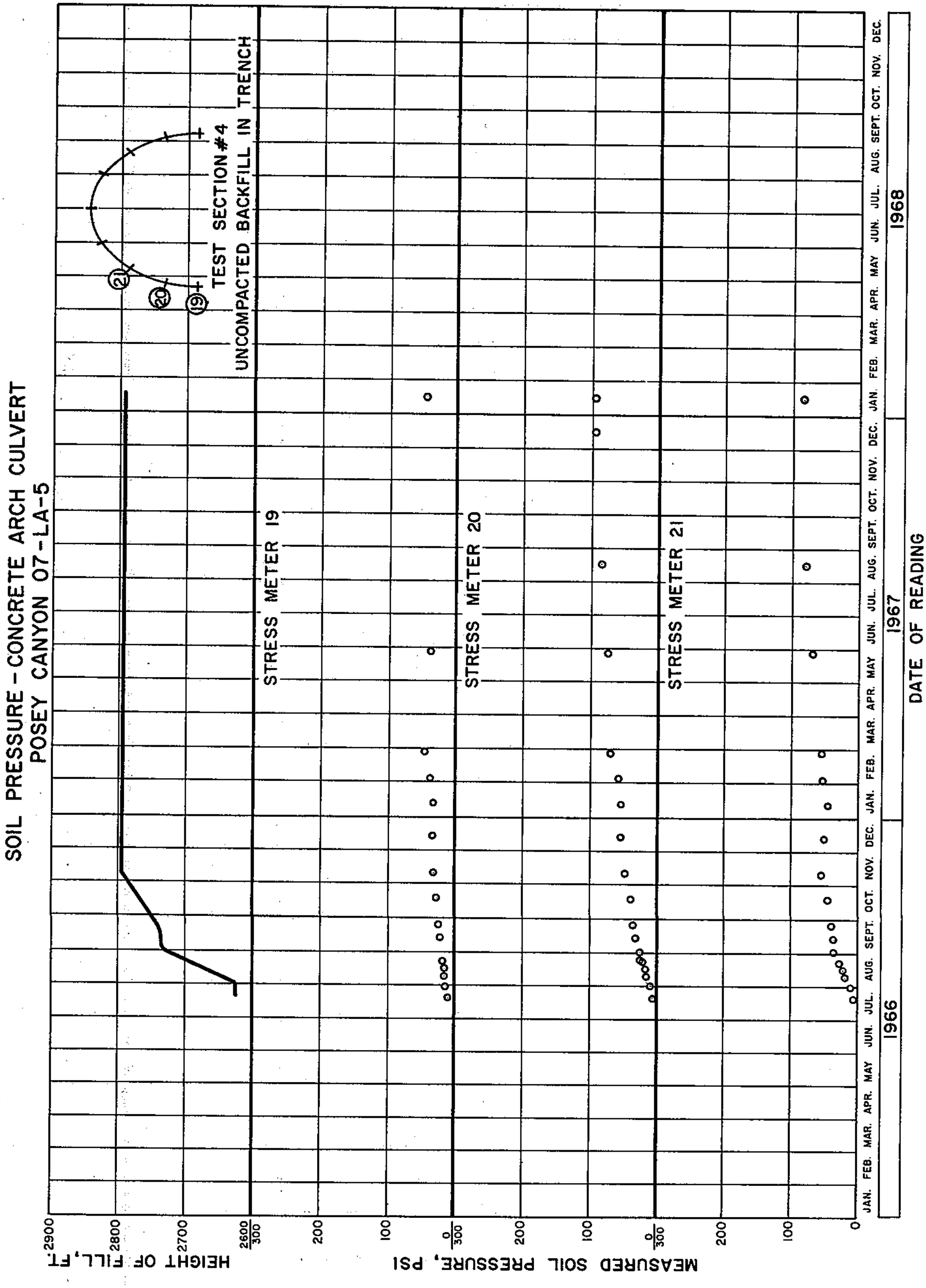
SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
HEIGHT OF FILL, FT.
2900
2800
2700
2600
300
200
100
0
300
200
100
0
300
200
100
0
MEASURED SOIL PRESSURE, PSI
TEST SECTION #2
COMPACTED SAND BACKFILL
13
14
15
STRESS METER 13
STRESS METER 14
STRESS METER 15
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 23



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #2
COMPACTED SAND BACKFILL
16
17
18
STRESS METER 16
STRESS METER 17
STRESS METER 18
HEIGHT OF FILL, FT.
MEASURED SOIL PRESSURE, PSI
2900
2800
2700
2600
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 24



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #4
UNCOMPACTED BACKFILL IN TRENCH
19
20
21
STRESS METER 19
STRESS METER 20
STRESS METER 21
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5

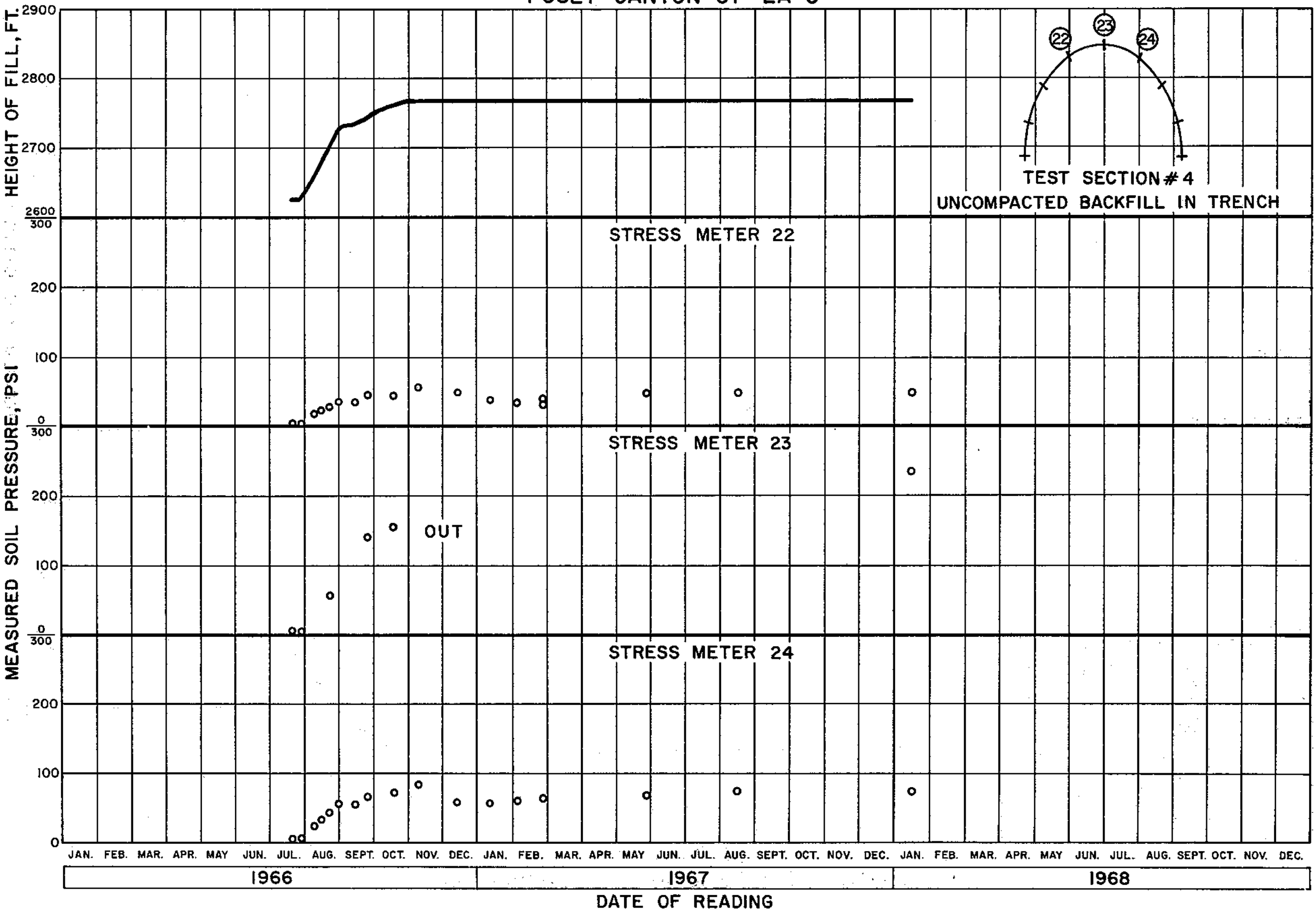
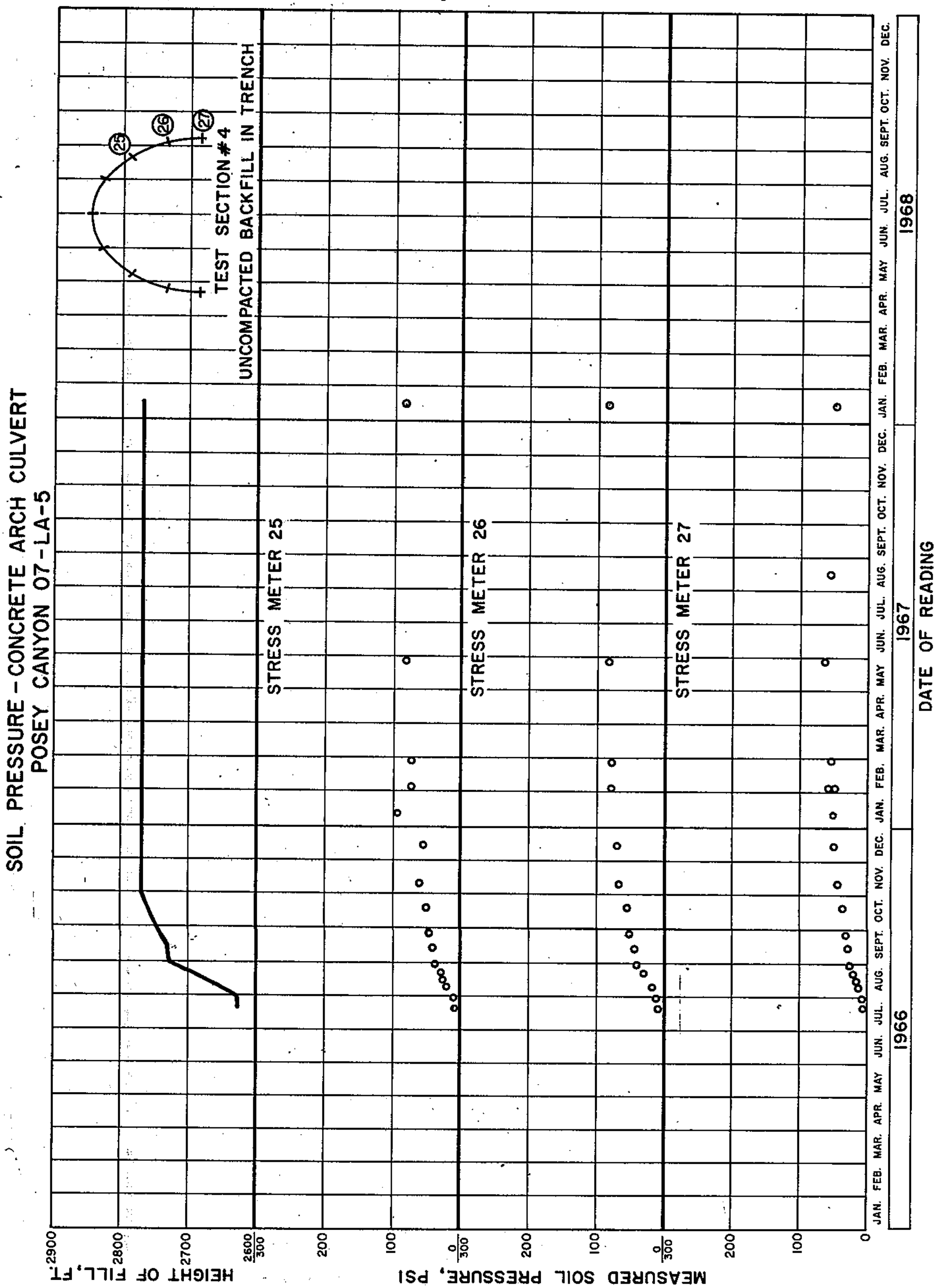


Figure 25

Figure 26



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION # 4
UNCOMPACTED BACKFILL IN TRENCH
25
26
27
STRESS METER 25
STRESS METER 26
STRESS METER 27
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 27

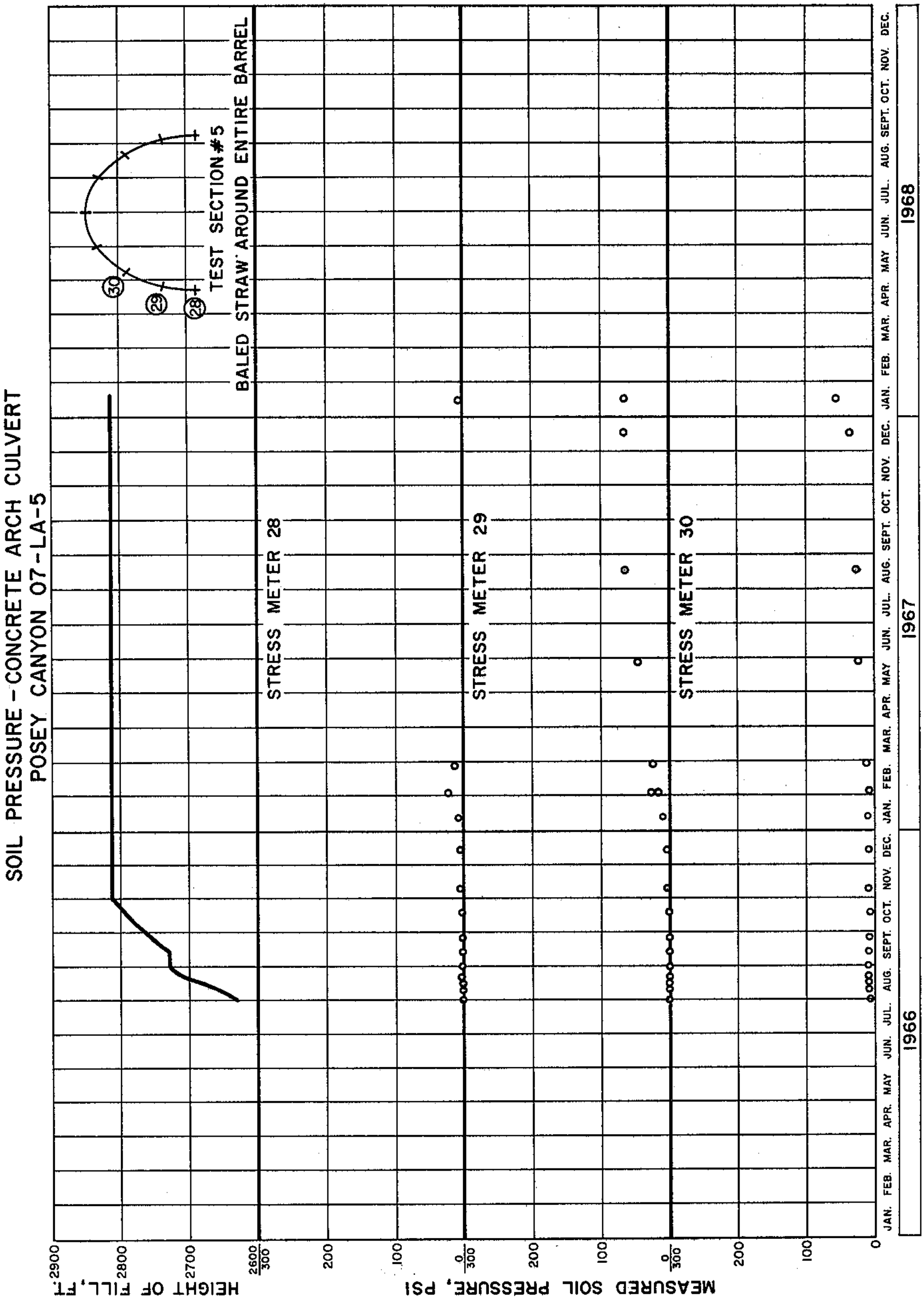

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #5
BALED STRAW AROUND ENTIRE BARREL
30
29
28
STRESS METER 28
STRESS METER 29
STRESS METER 30
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
DATE OF READING
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968

Figure 28

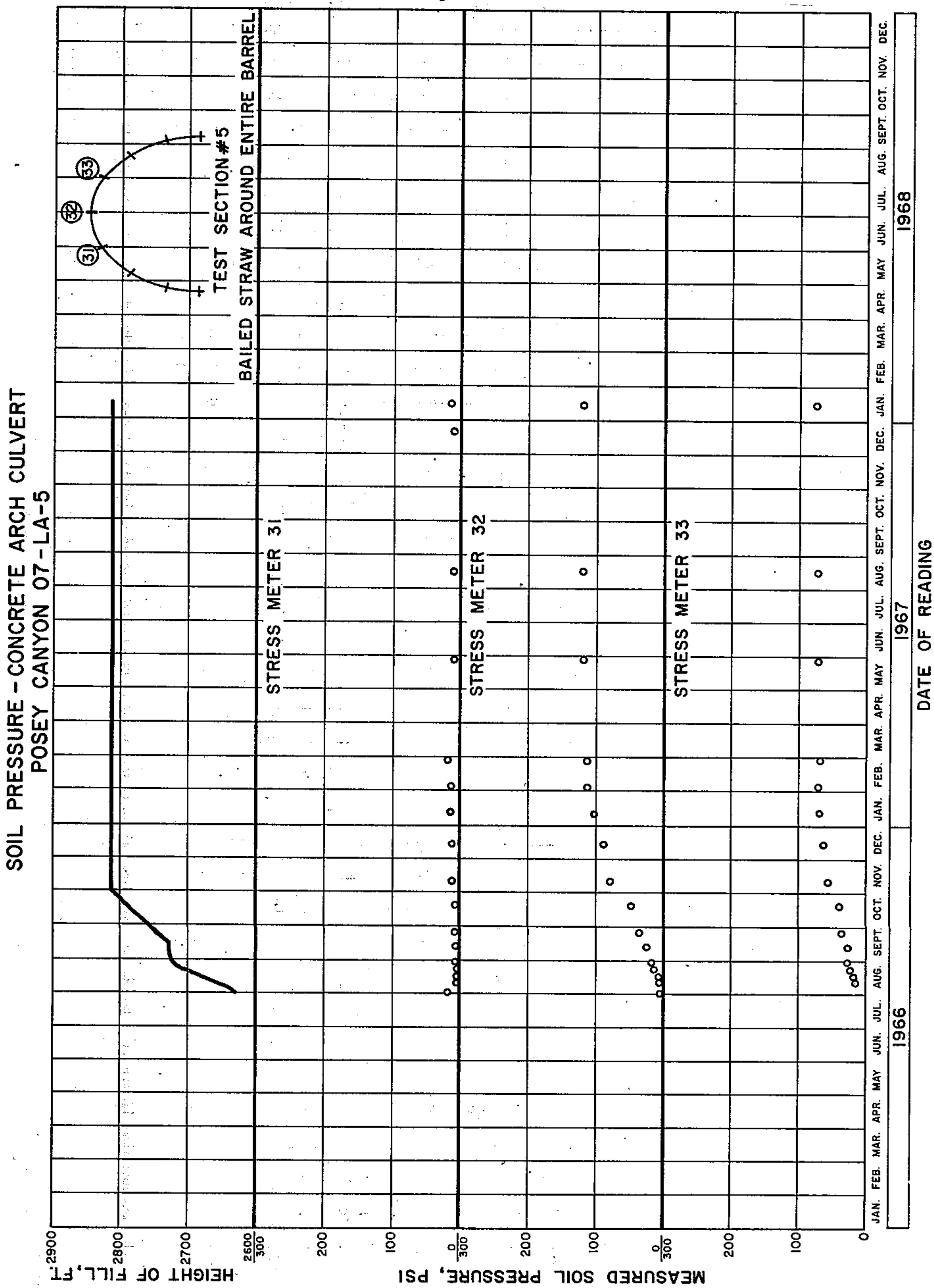
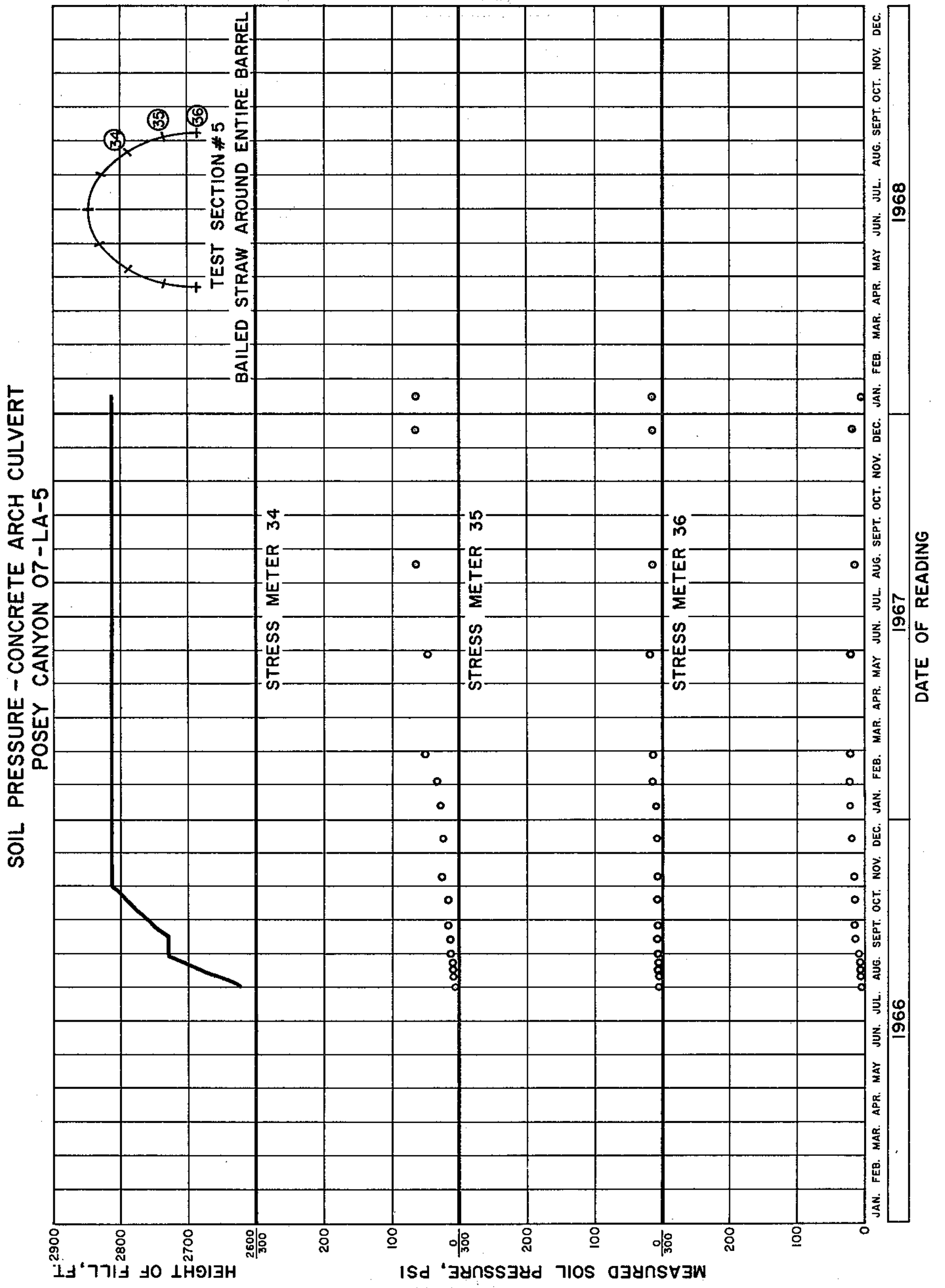
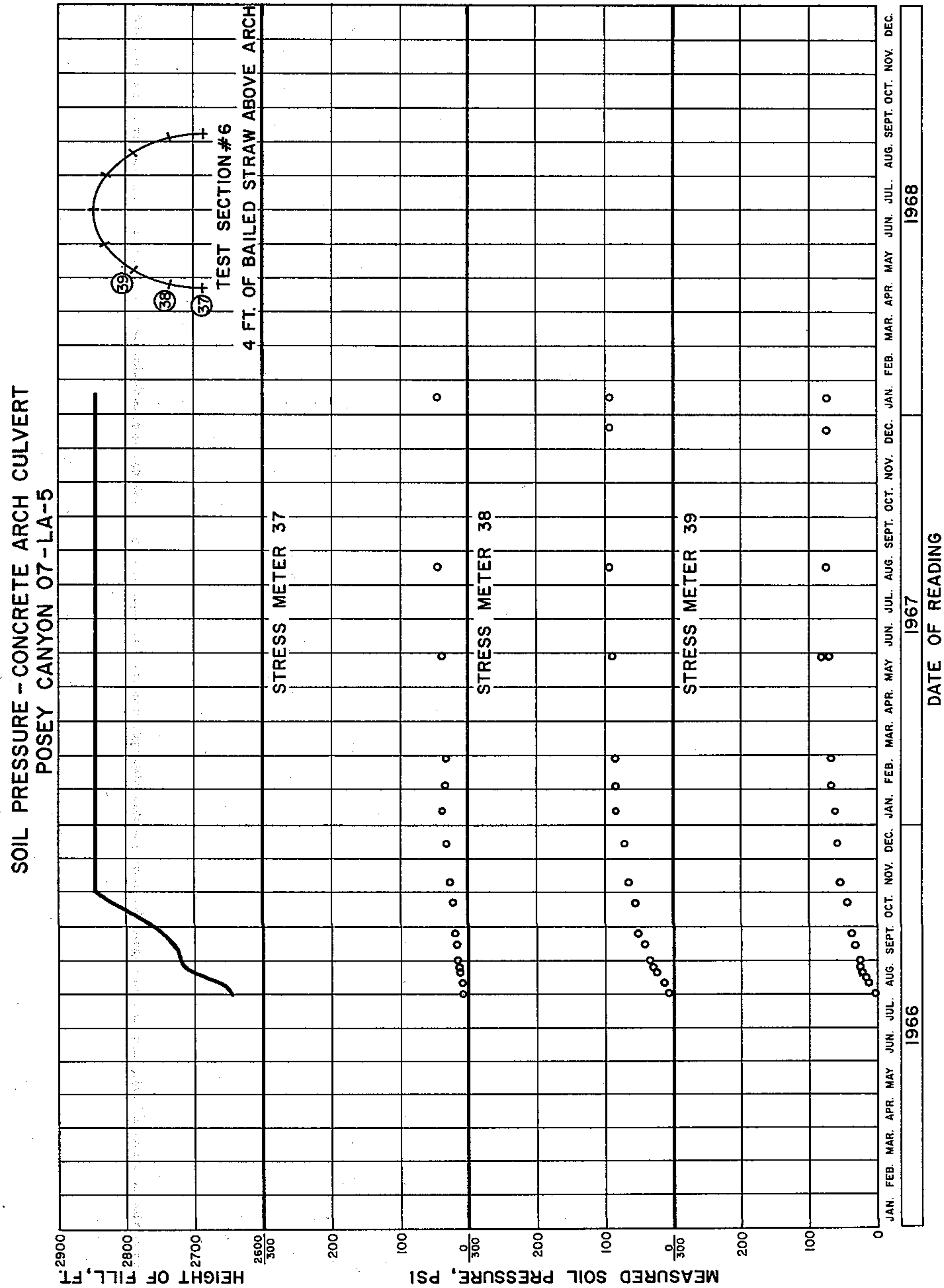

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #5
BAILED STRAW AROUND ENTIRE BARREL
31
32
33
STRESS METER 31
STRESS METER 32
STRESS METER 33
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 29



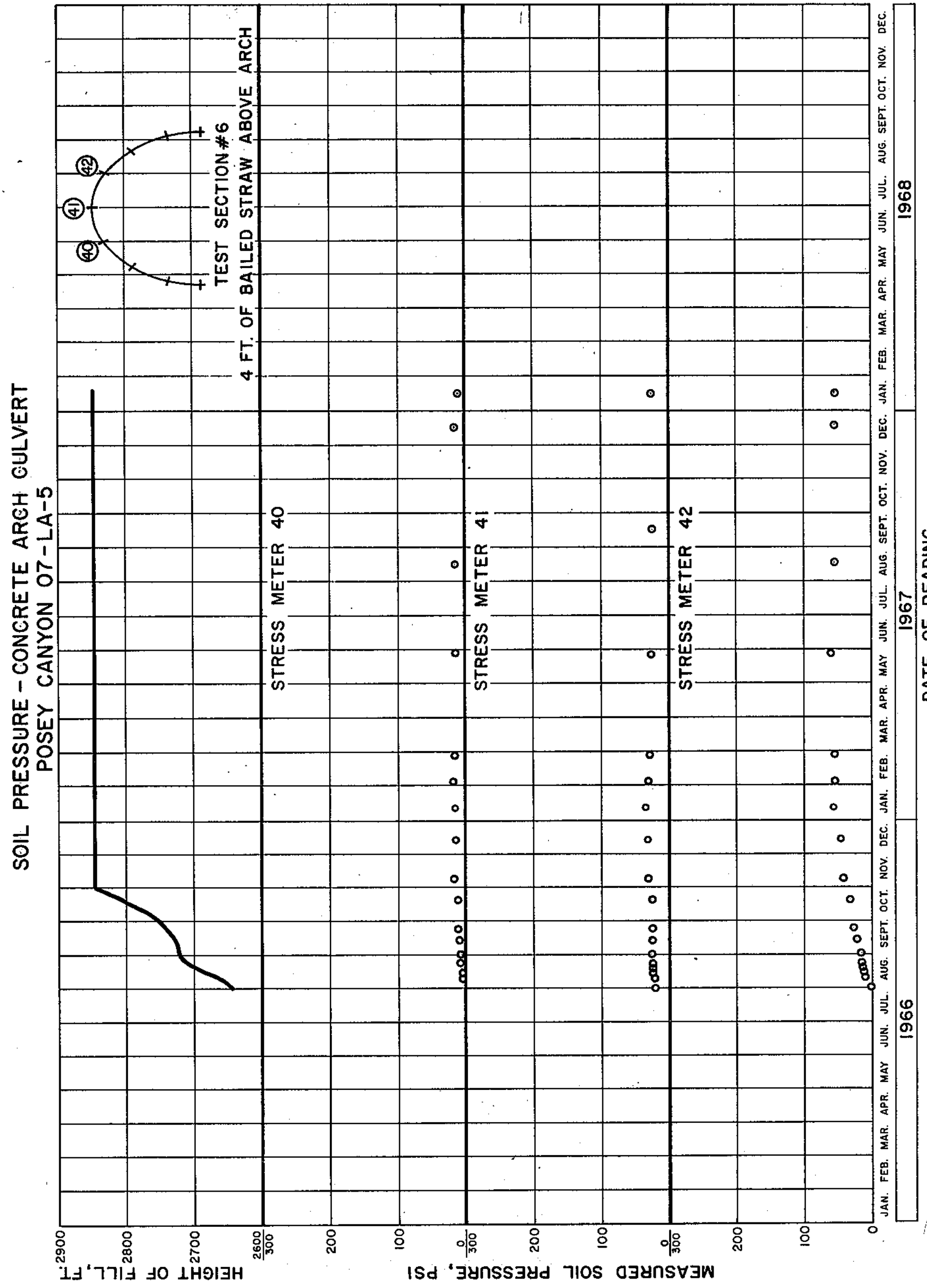
SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION # 5
BAILED STRAW AROUND ENTIRE BARREL
34
35
36
STRESS METER 34
STRESS METER 35
STRESS METER 36
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 30



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #6
4 FT. OF BAILED STRAW ABOVE ARCH
STRESS METER 37
STRESS METER 38
STRESS METER 39
HEIGHT OF FILL, FT.
MEASURED SOIL PRESSURE, PSI
DATE OF READING
1966
1967
1968
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
2900
2800
2700
2600
300
200
100
0

Figure 31



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #6
4 FT. OF BAILED STRAW ABOVE ARCH
40
41
42
STRESS METER 40
STRESS METER 41
STRESS METER 42
HEIGHT OF FILL, FT.
MEASURED SOIL PRESSURE, PSI
2900
2800
2700
2600
300
200
100
0
DATE OF READING
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968

Figure 32

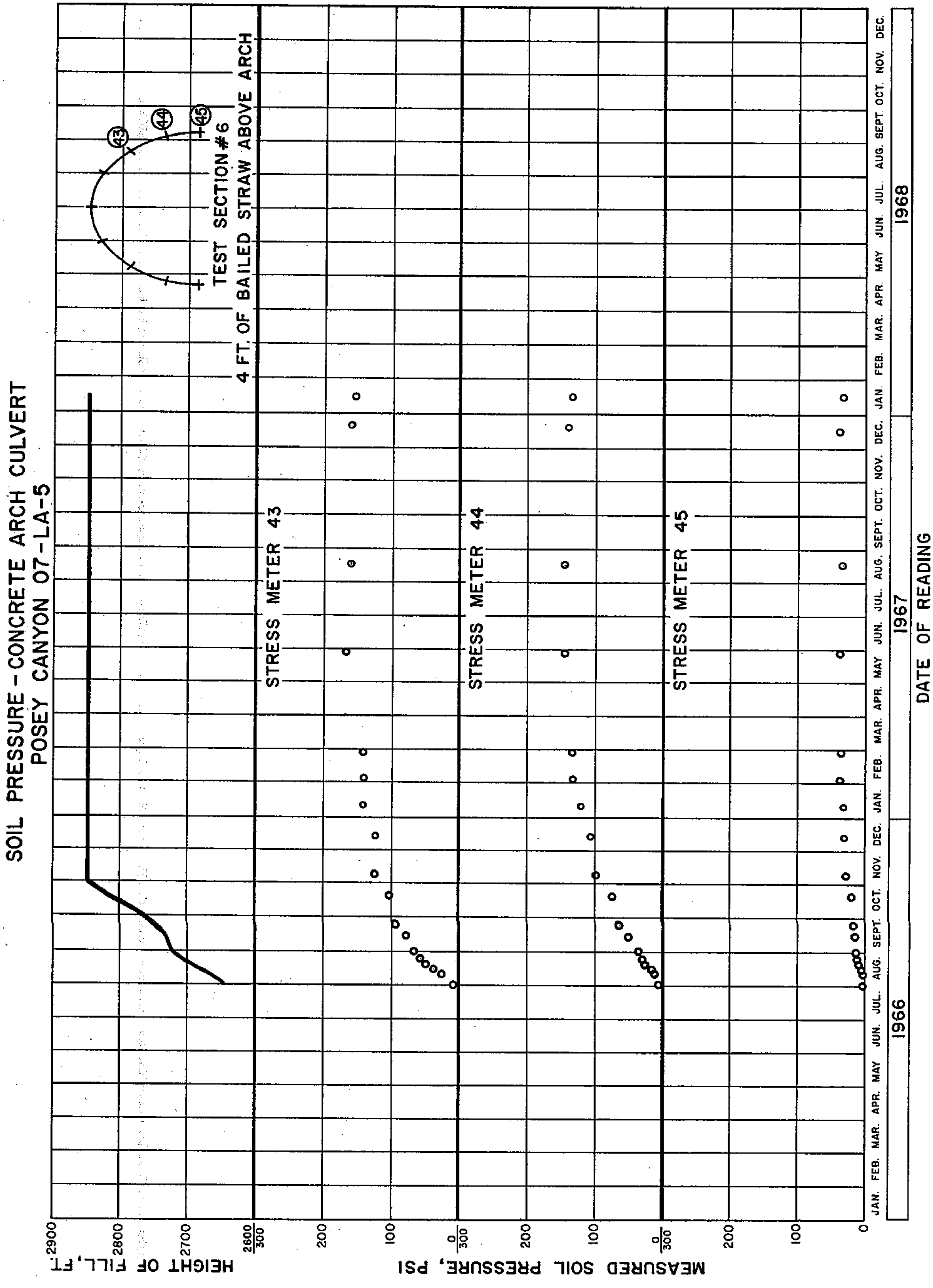

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #6
4 FT. OF BAILED STRAW ABOVE ARCH
STRESS METER 43
STRESS METER 44
STRESS METER 45
HEIGHT OF FILL, FT.
MEASURED SOIL PRESSURE, PSI
DATE OF READING
1966
1967
1968
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
2900
2800
2700
2600
300
200
100
0
43
44
45

Figure 33

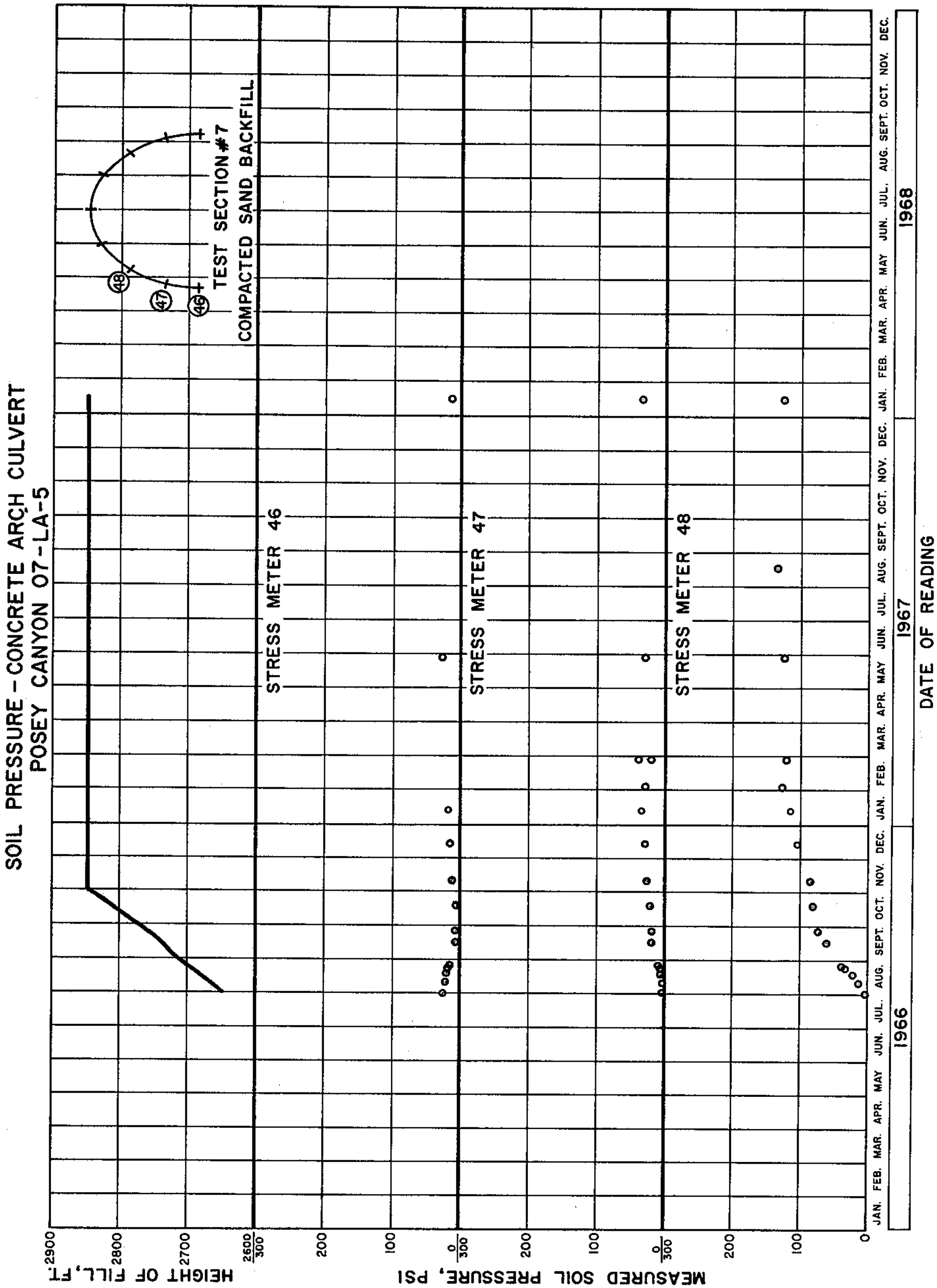
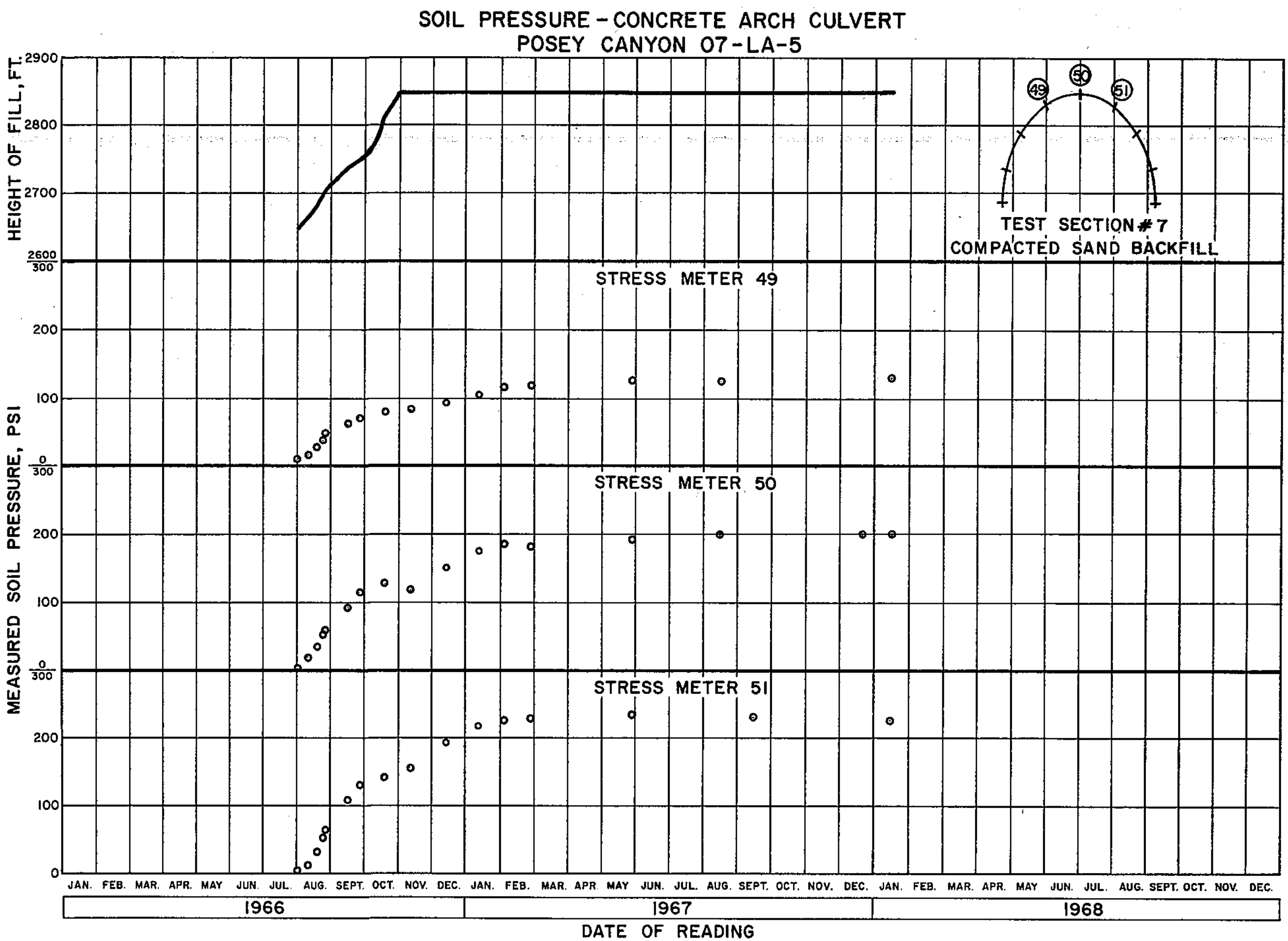
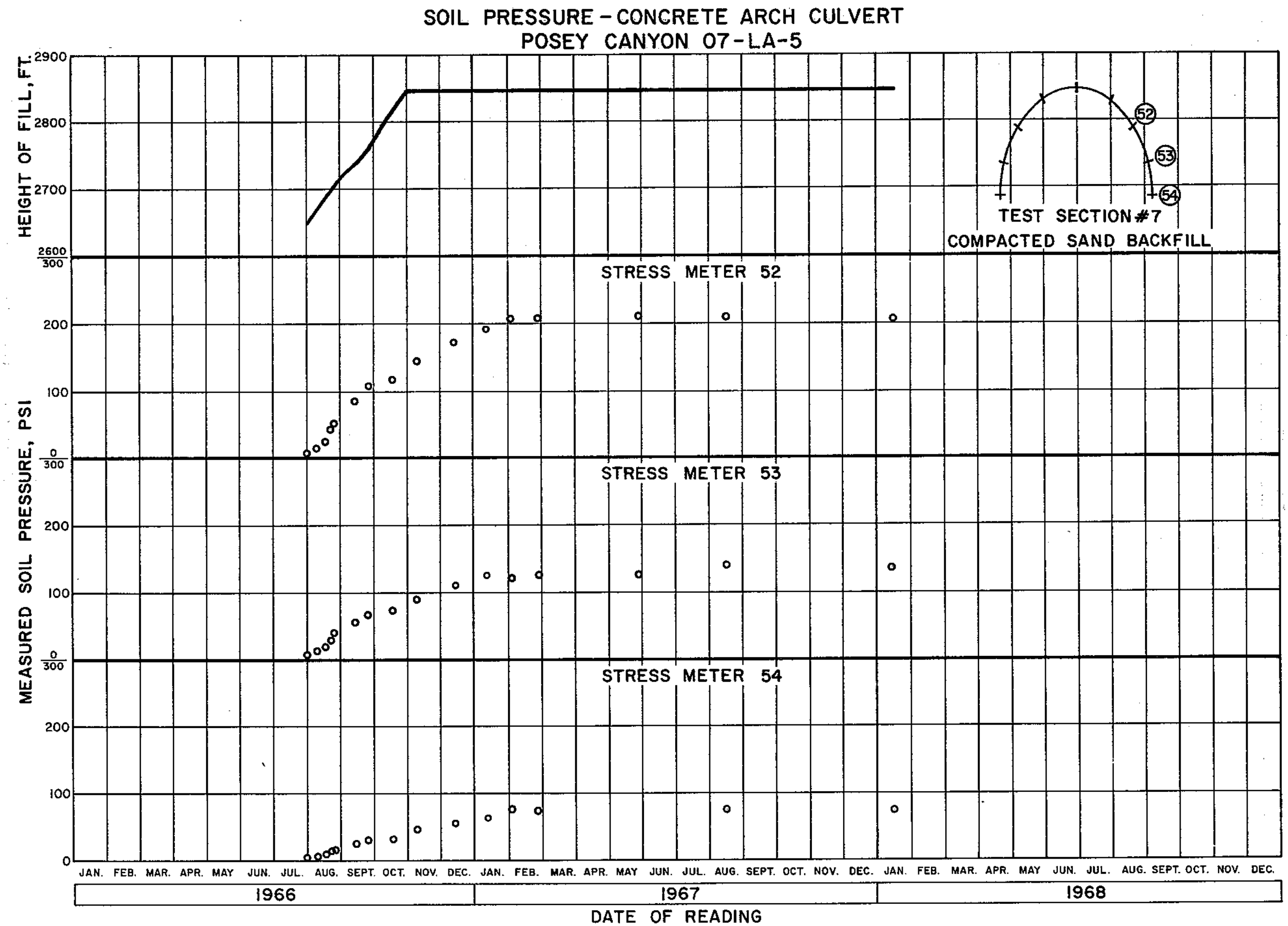

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #7
COMPACTED SAND BACKFILL
46
47
48
STRESS METER 46
STRESS METER 47
STRESS METER 48
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 34



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
TEST SECTION #7
COMPACTED SAND BACKFILL
49
50
51
STRESS METER 49
STRESS METER 50
STRESS METER 51
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 35



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
HEIGHT OF FILL, FT.
2900
2800
2700
2600
TEST SECTION #7
COMPACTED SAND BACKFILL
52
53
54
STRESS METER 52
STRESS METER 53
STRESS METER 54
MEASURED SOIL PRESSURE, PSI
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 36

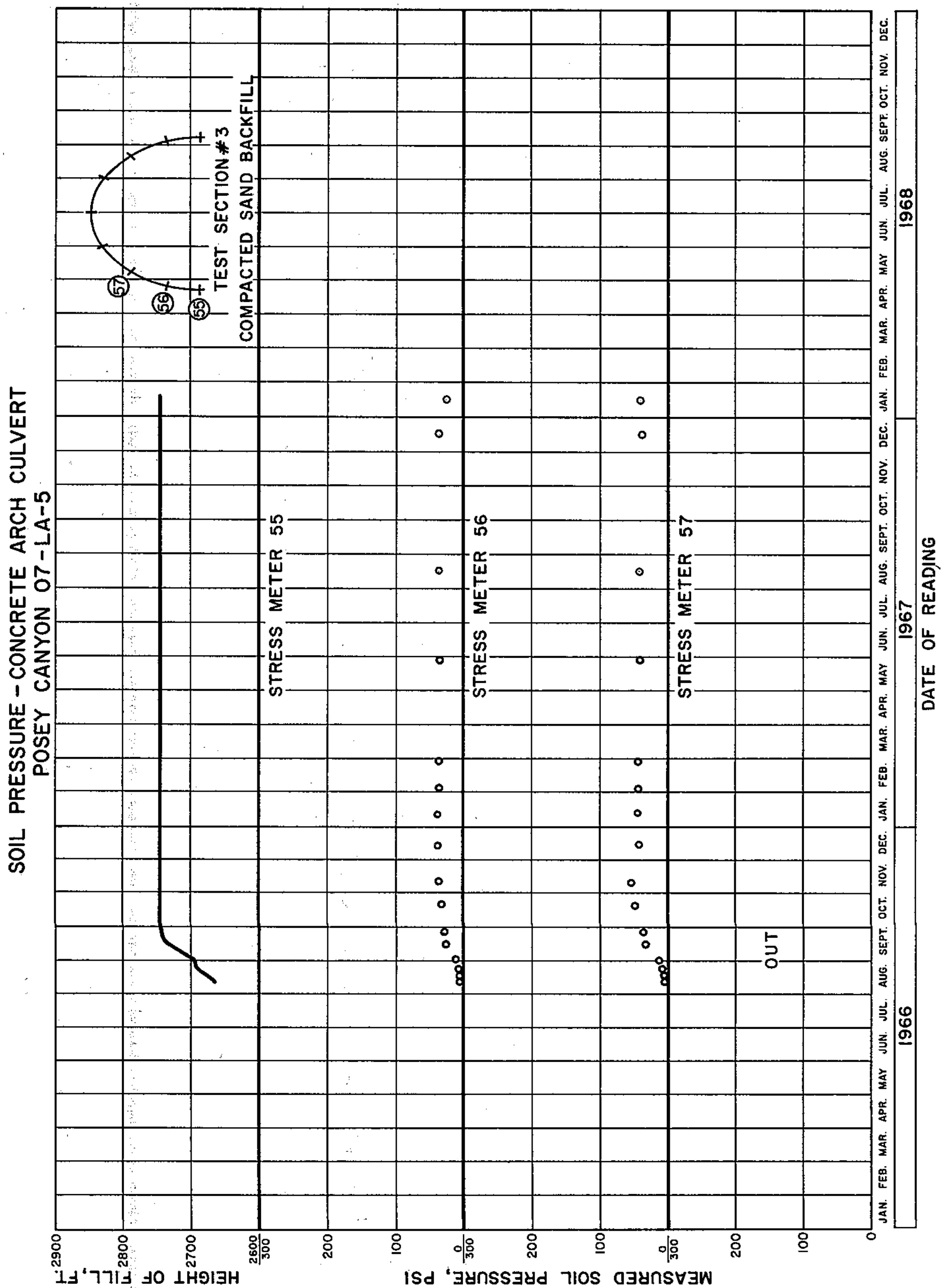
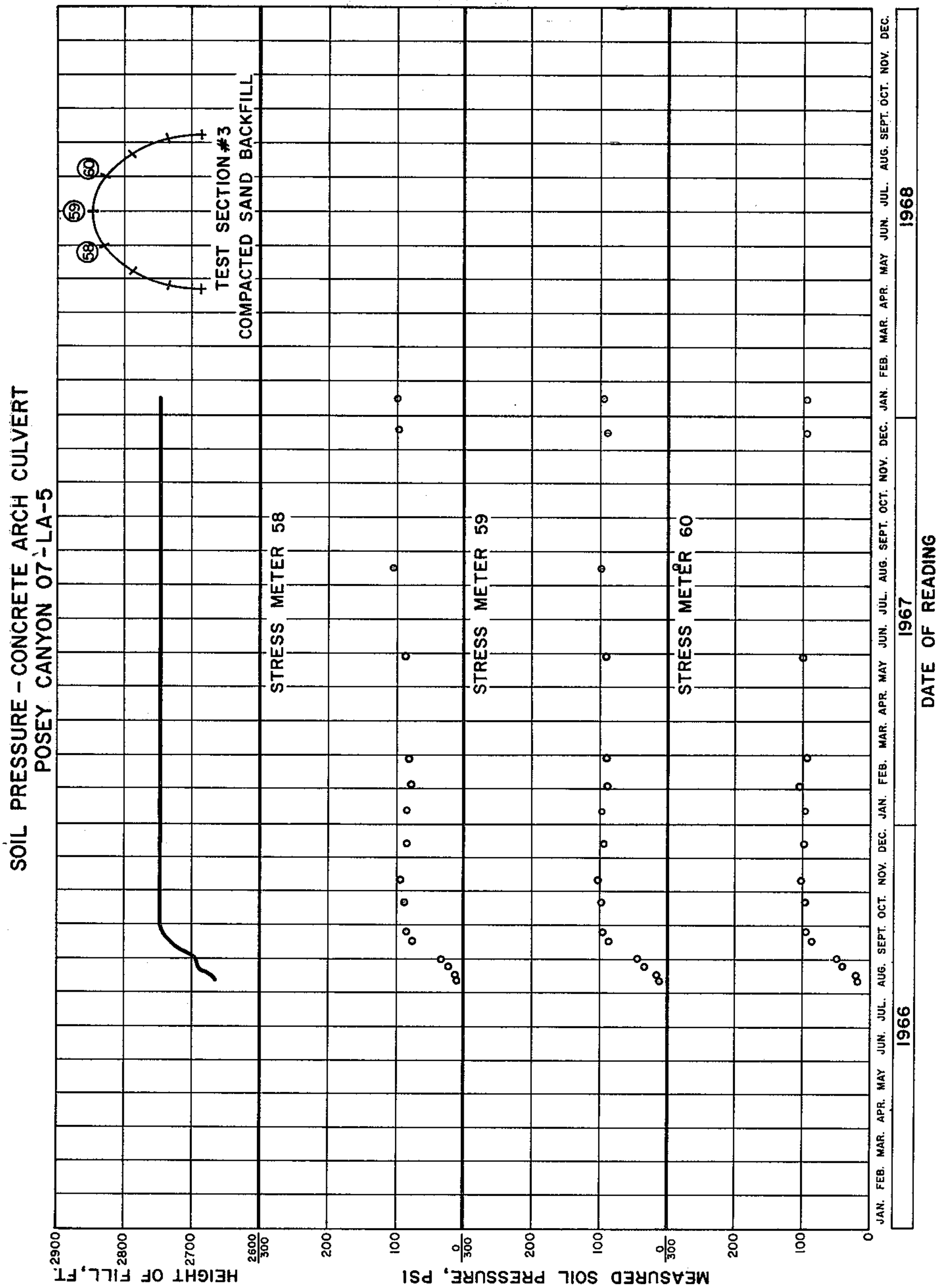

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #3
COMPACTED SAND BACKFILL
STRESS METER 55
STRESS METER 56
STRESS METER 57
OUT
HEIGHT OF FILL, FT.
MEASURED SOIL PRESSURE, PSI
DATE OF READING
1966
1967
1968

Figure 37



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #3
COMPACTED SAND BACKFILL
58
59
60
STRESS METER 58
STRESS METER 59
STRESS METER 60
HEIGHT OF FILL, FT.
2900
2800
2700
2600
MEASURED SOIL PRESSURE, PSI
300
200
100
0
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 38

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5

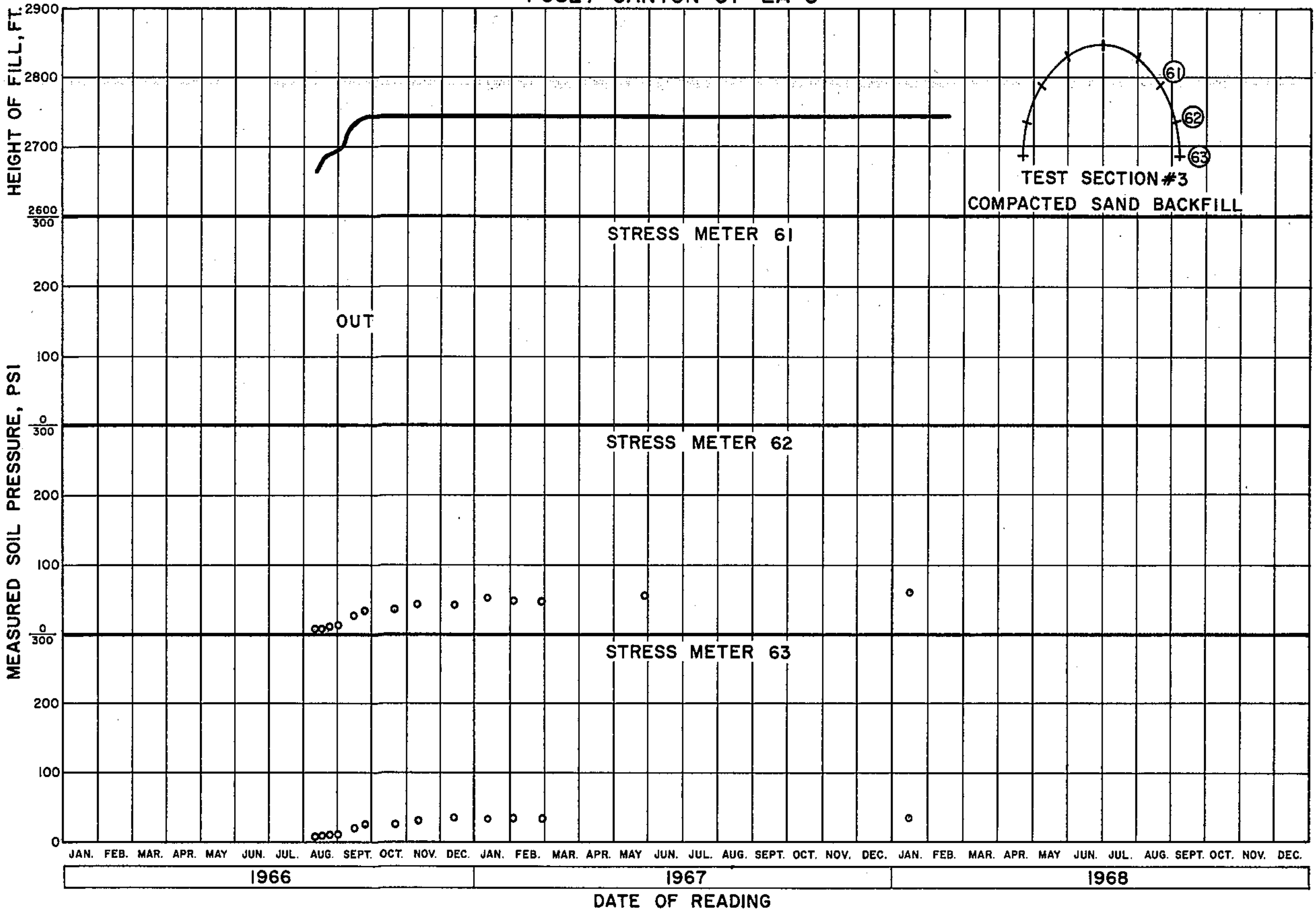
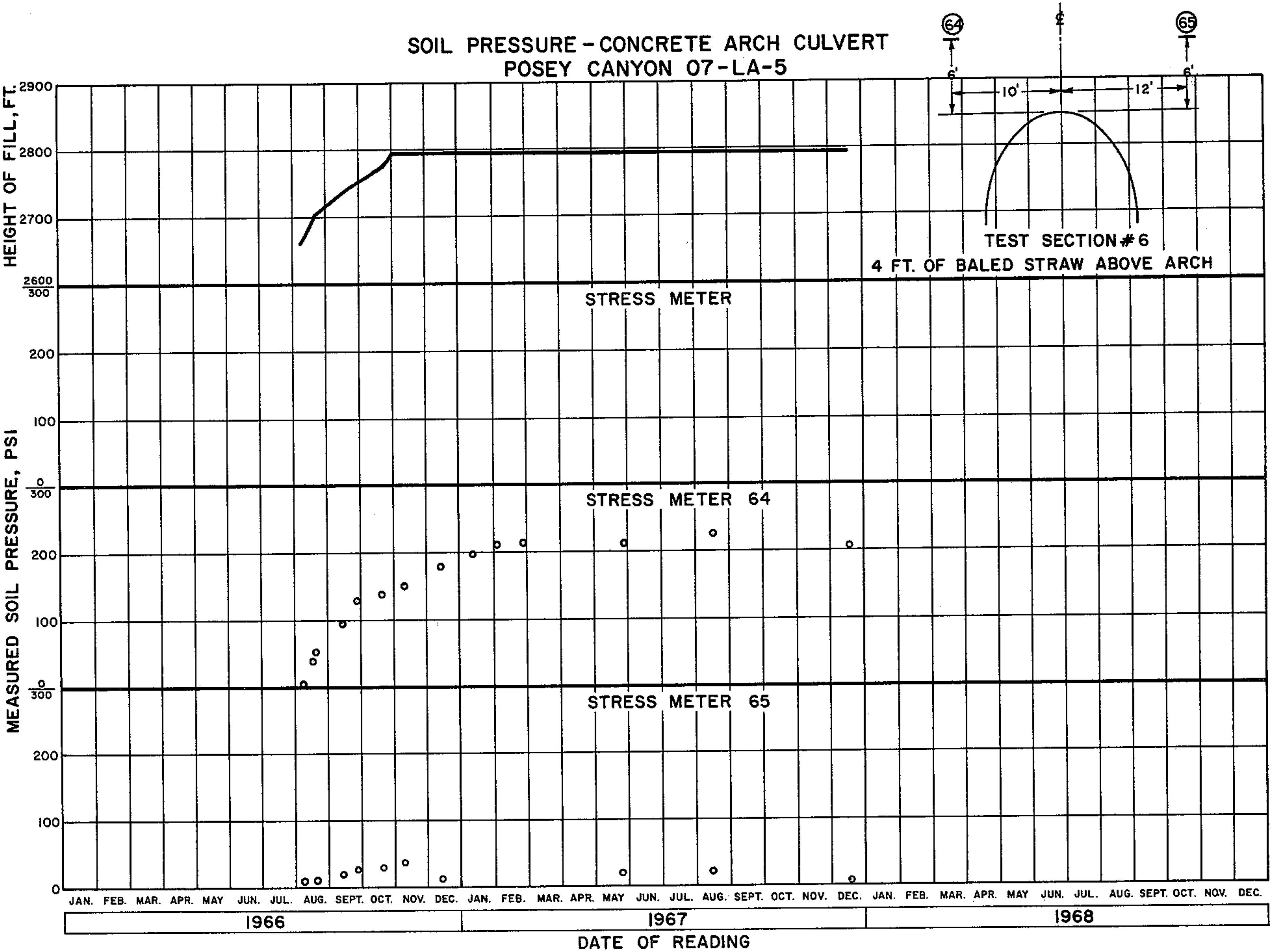
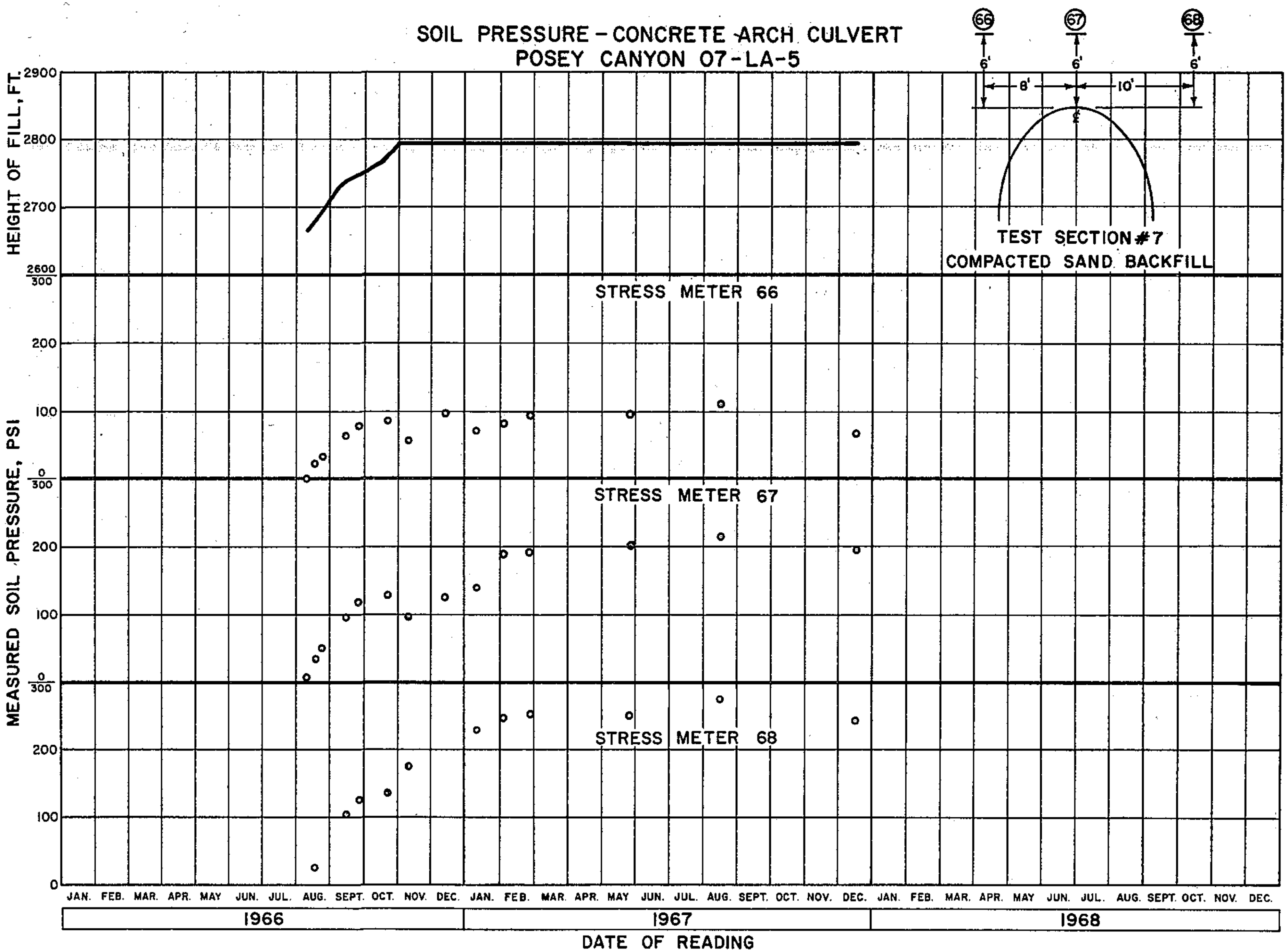


Figure 39



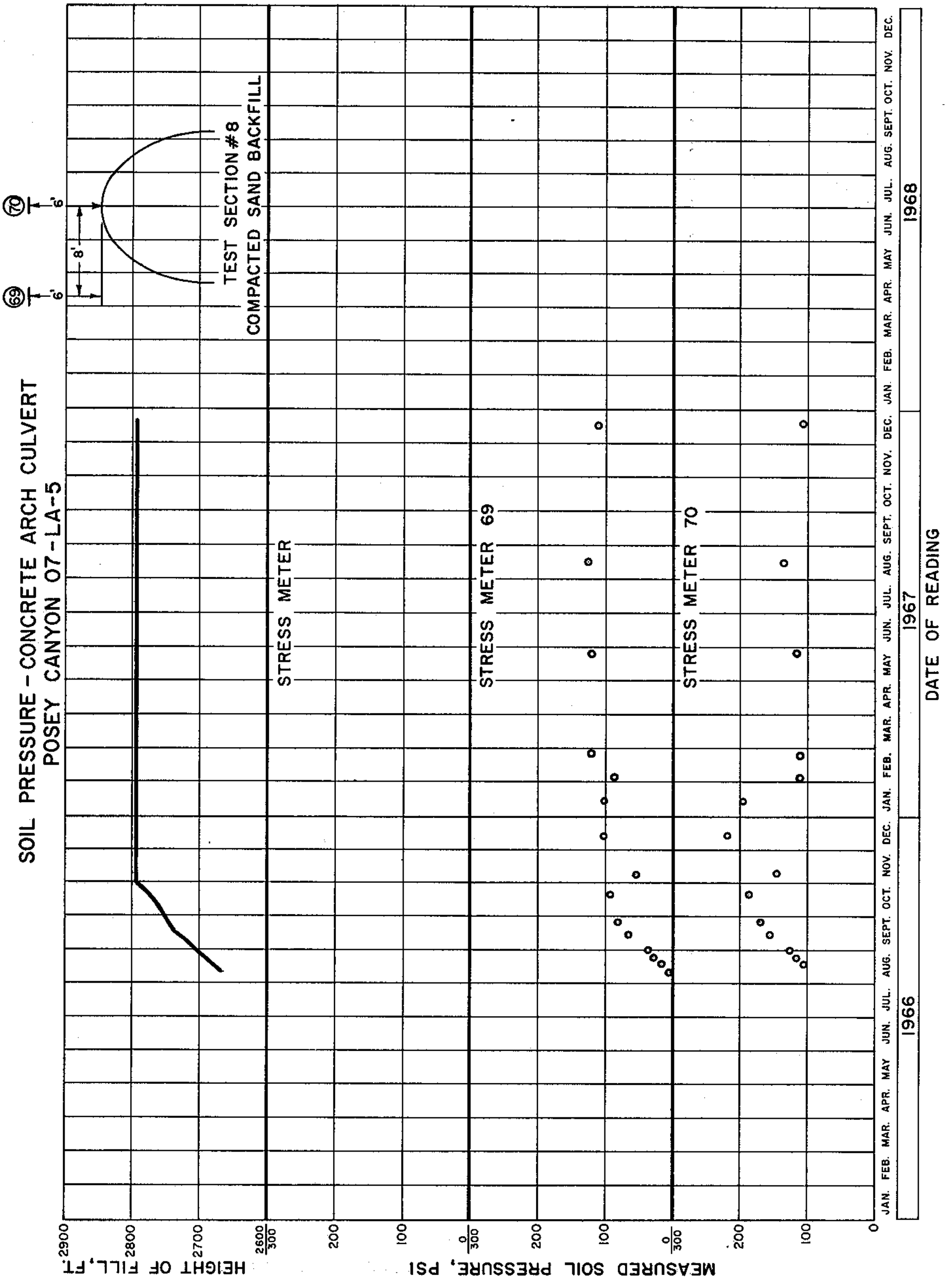
SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
64
65
6'
10'
12'
6'
TEST SECTION # 6
4 FT. OF BALED STRAW ABOVE ARCH
HEIGHT OF FILL, FT.
2900
2800
2700
2600
300
STRESS METER
200
100
0
300
STRESS METER 64
200
100
0
300
STRESS METER 65
200
100
0
MEASURED SOIL PRESSURE, PSI
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 40

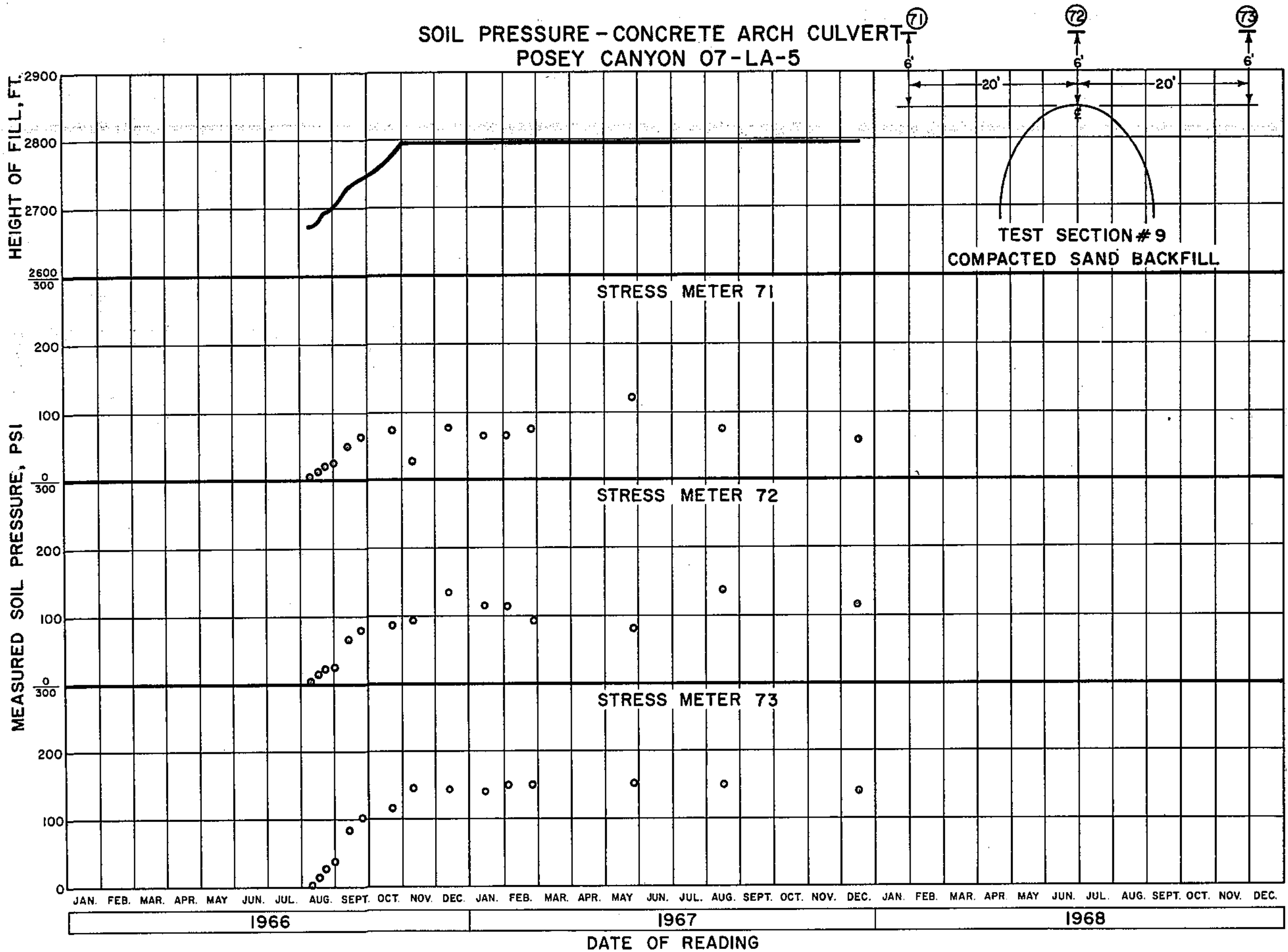


SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
66
67
68
6'
8'
10'
TEST SECTION #7
COMPACTED SAND BACKFILL
HEIGHT OF FILL, FT.
2900
2800
2700
2600
300
200
100
0
STRESS METER 66
STRESS METER 67
STRESS METER 68
MEASURED SOIL PRESSURE, PSI
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 41



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
TEST SECTION #8
COMPACTED SAND BACKFILL
69
70
6'
8'
6'
STRESS METER
STRESS METER 69
STRESS METER 70
HEIGHT OF FILL, FT.
MEASURED SOIL PRESSURE, PSI
DATE OF READING
1966
1967
1968
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
2900
2800
2700
2600
300
200
100
0



SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5
71
72
73
6'
20'
TEST SECTION # 9
COMPACTED SAND BACKFILL
HEIGHT OF FILL, FT.
2900
2800
2700
2600
300
200
100
0
MEASURED SOIL PRESSURE, PSI
STRESS METER 71
STRESS METER 72
STRESS METER 73
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEPT. OCT. NOV. DEC.
1966
1967
1968
DATE OF READING

Figure 42

Figure 43

SOIL PRESSURE - CONCRETE ARCH CULVERT
POSEY CANYON 07-LA-5

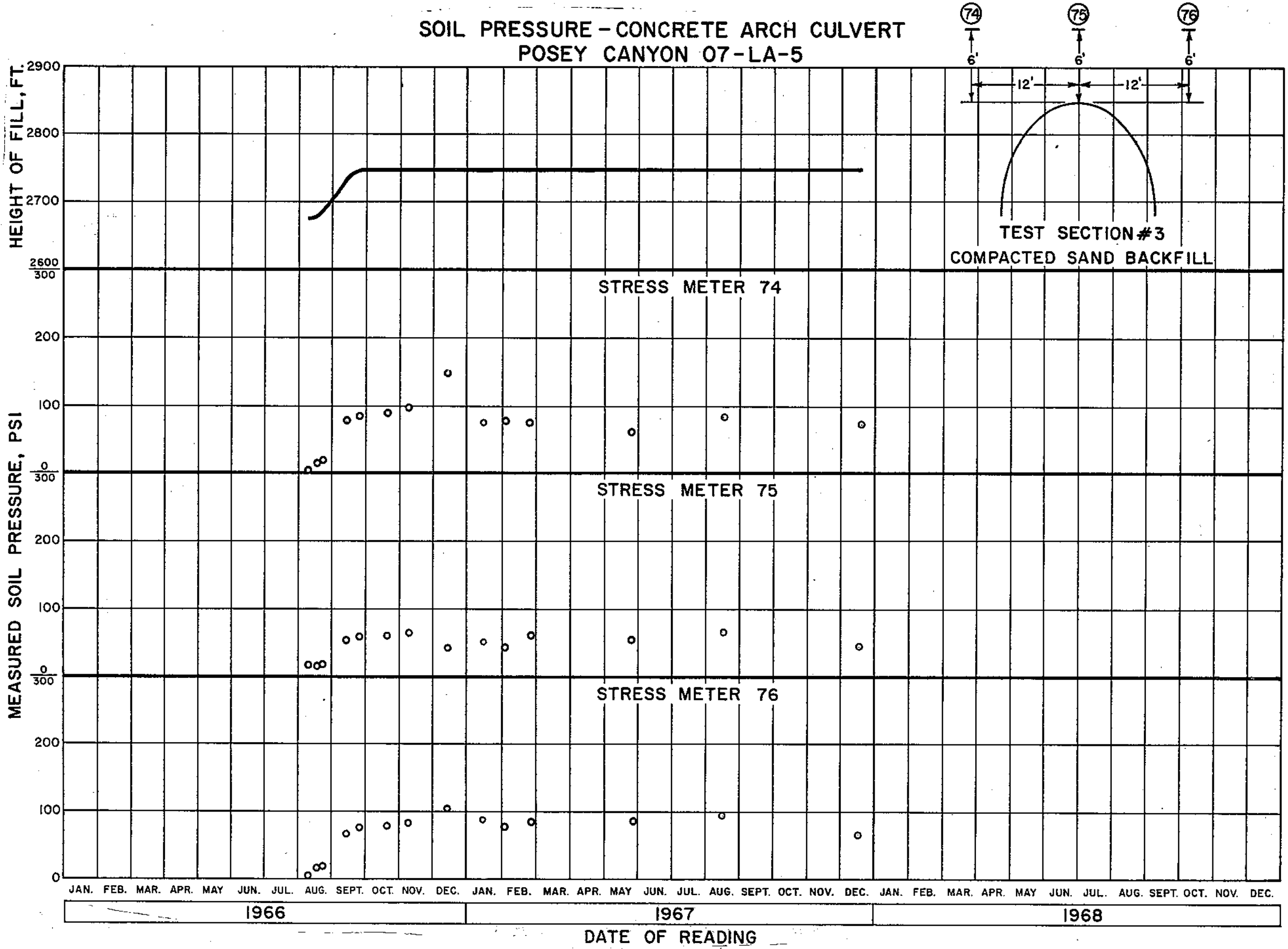


Figure 44

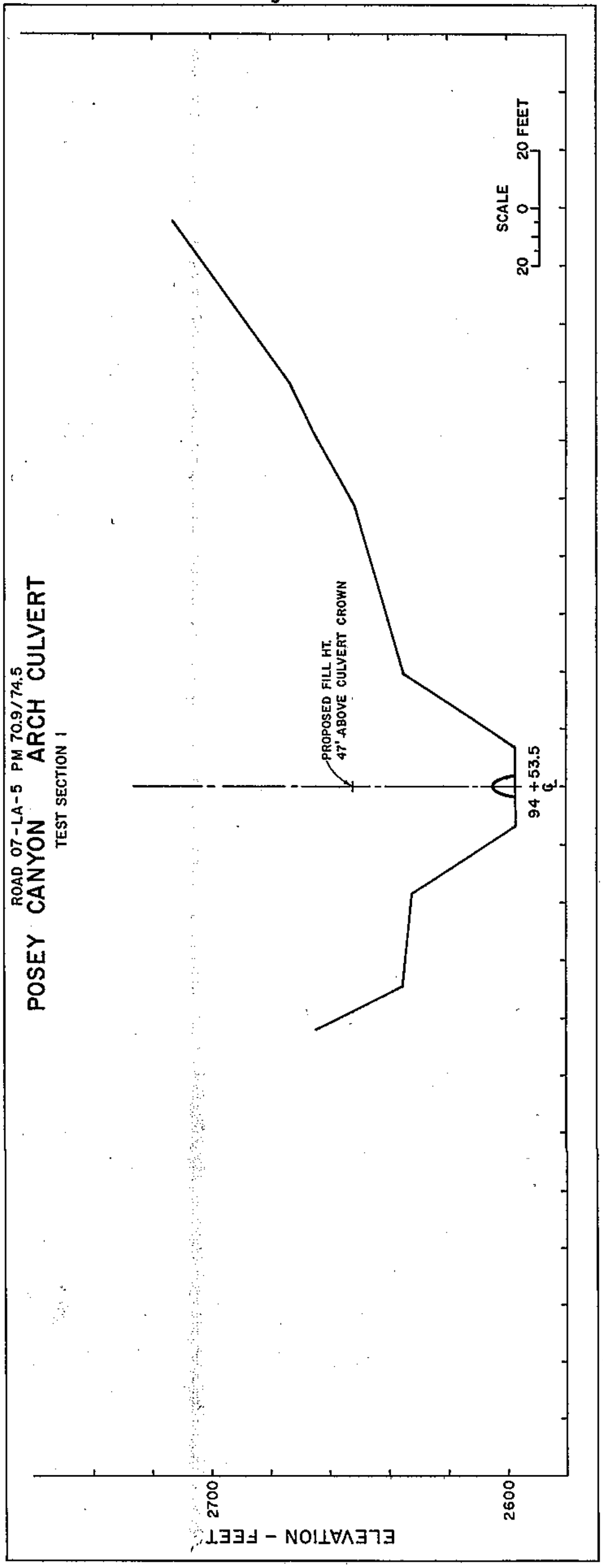


Figure 45

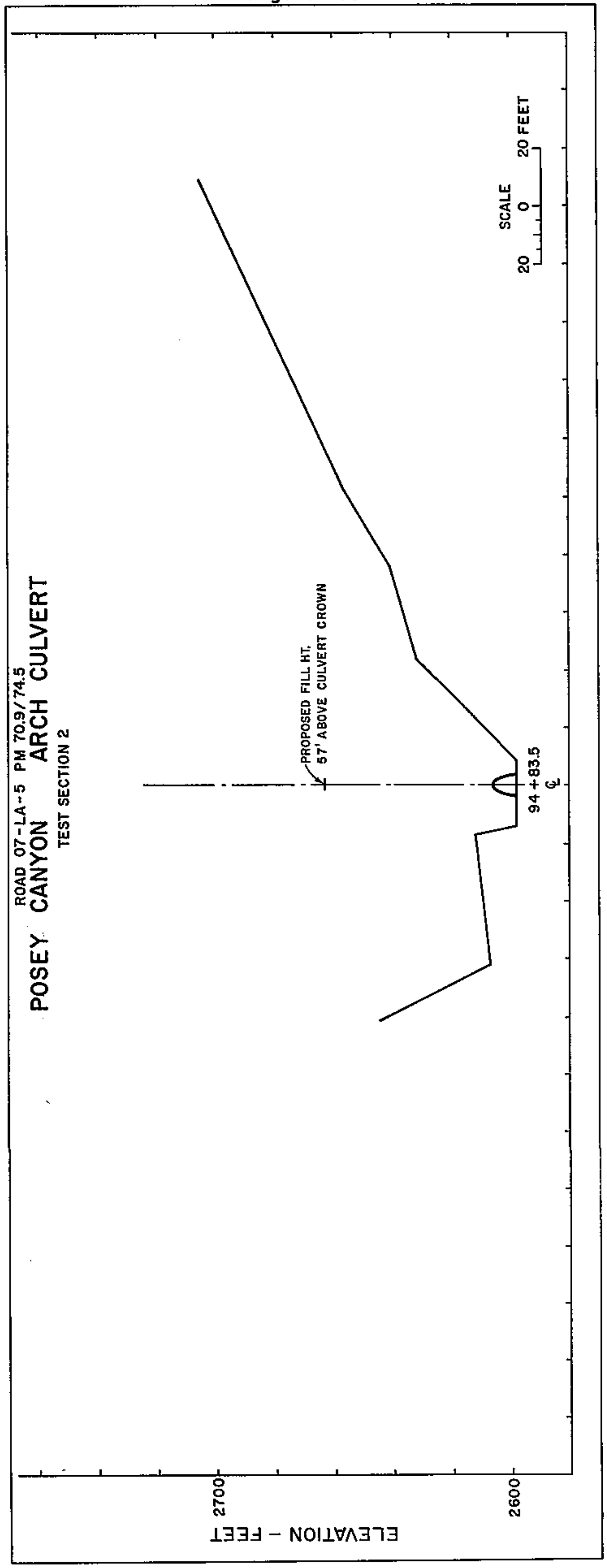


Figure 46

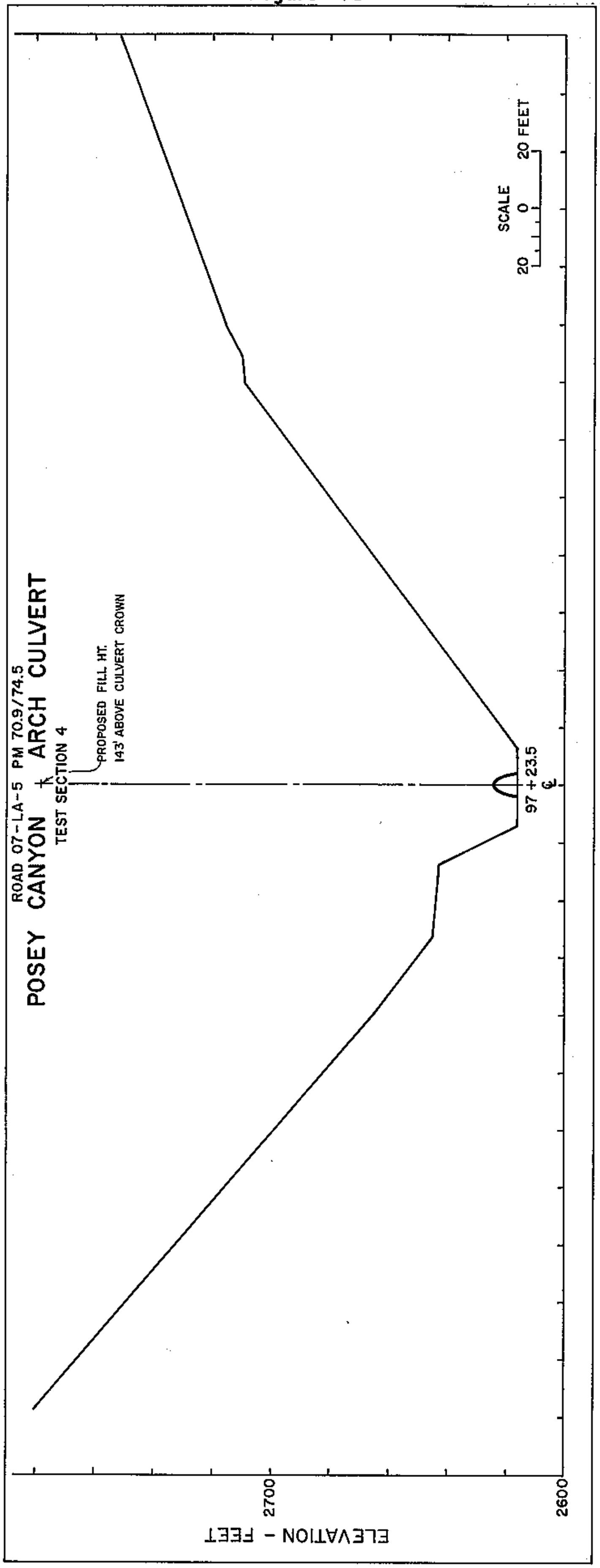


Figure 47

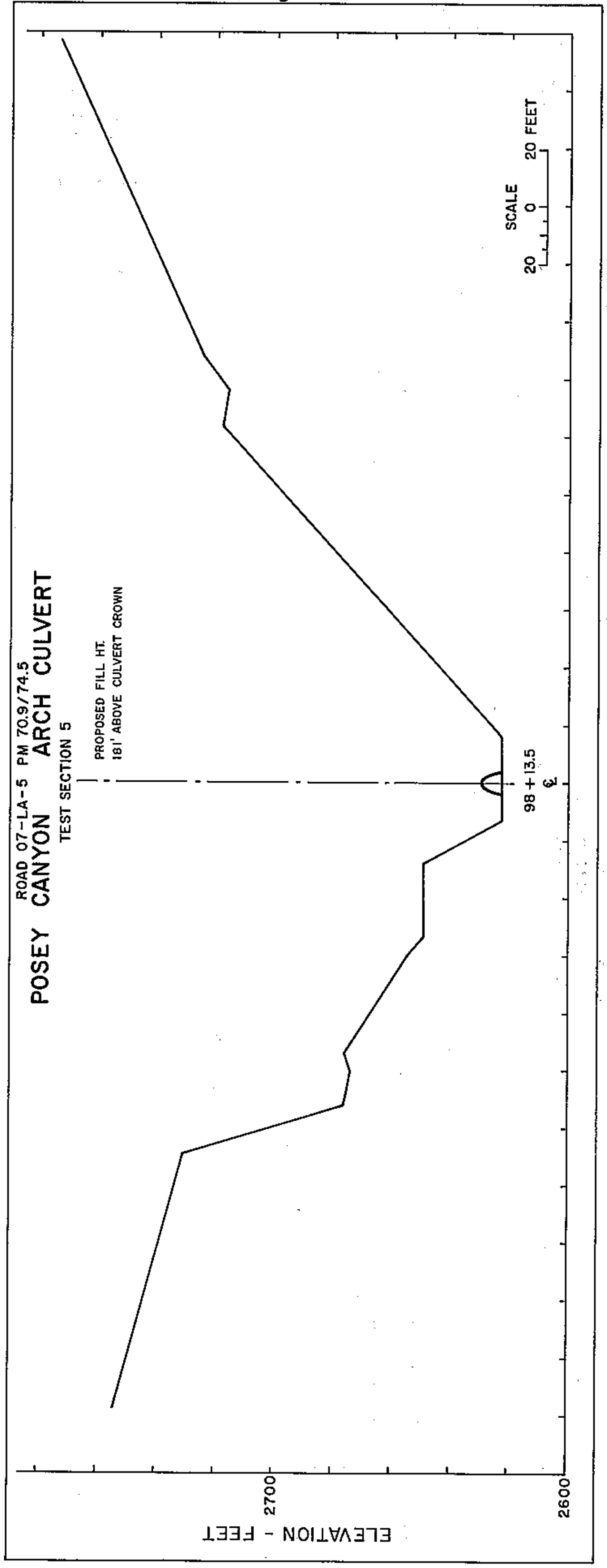


Figure 48

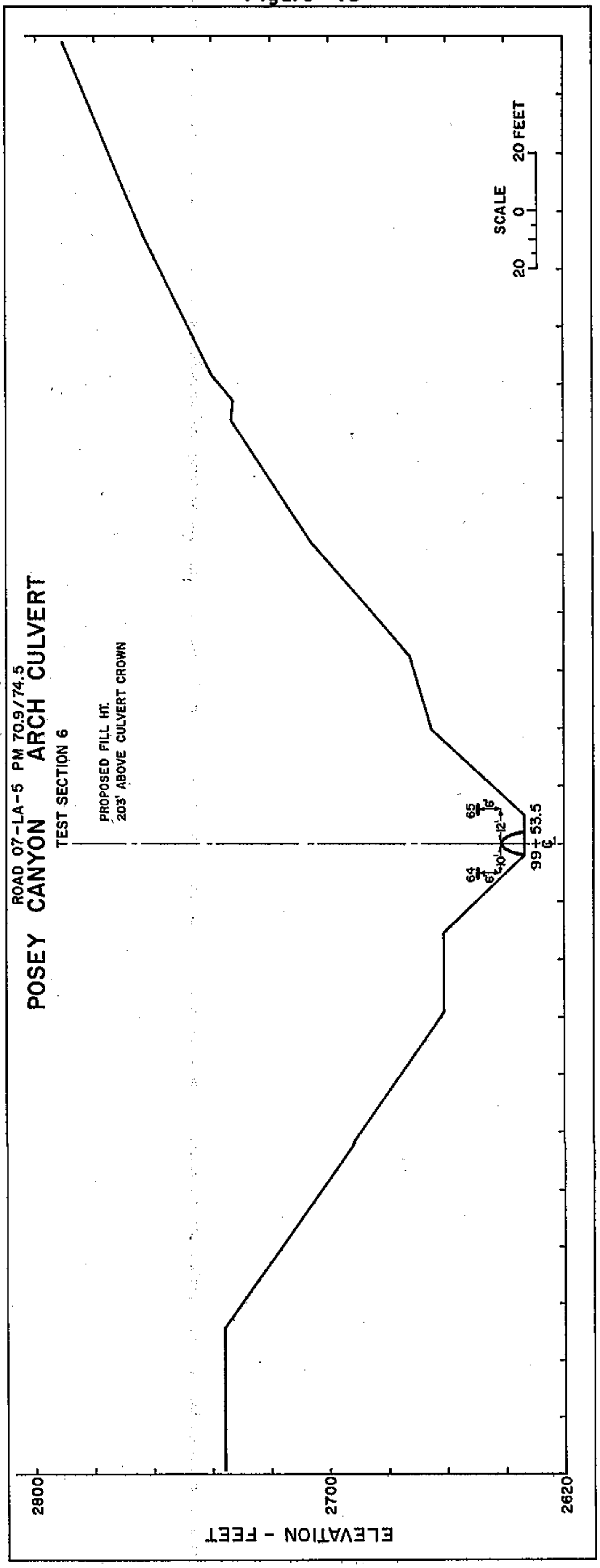


Figure 49

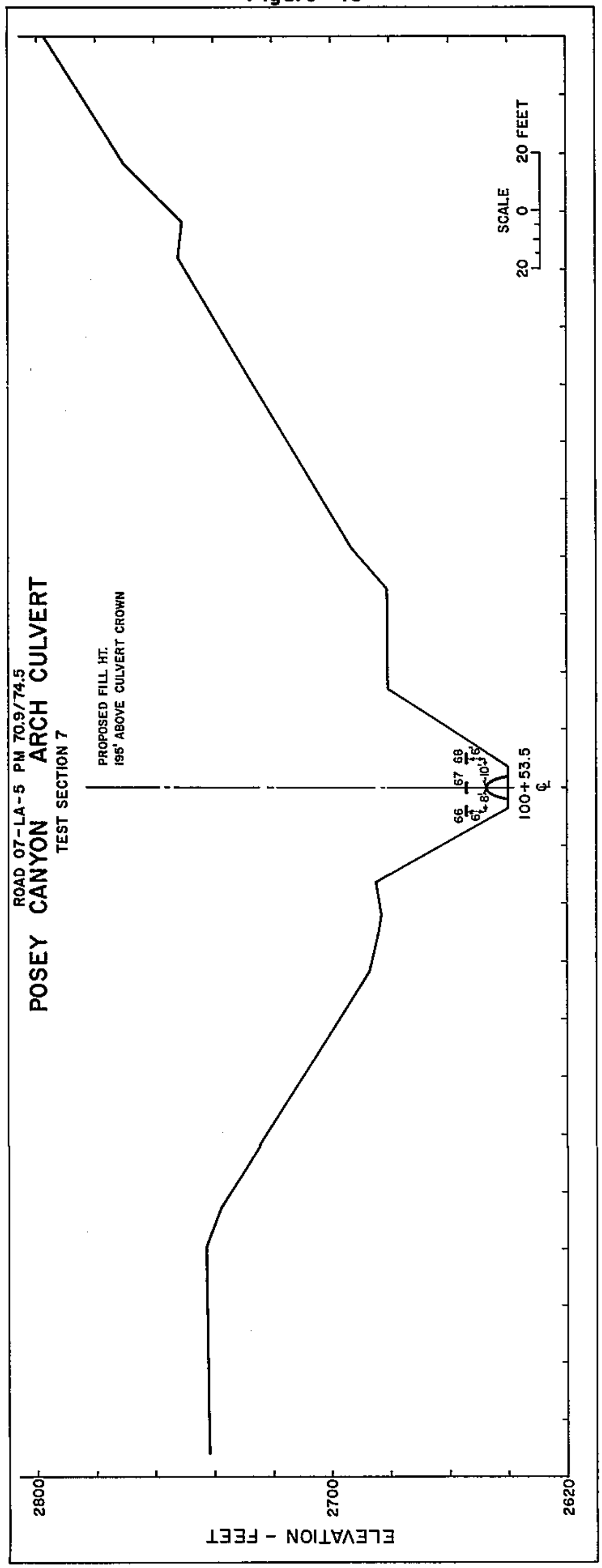


Figure 50

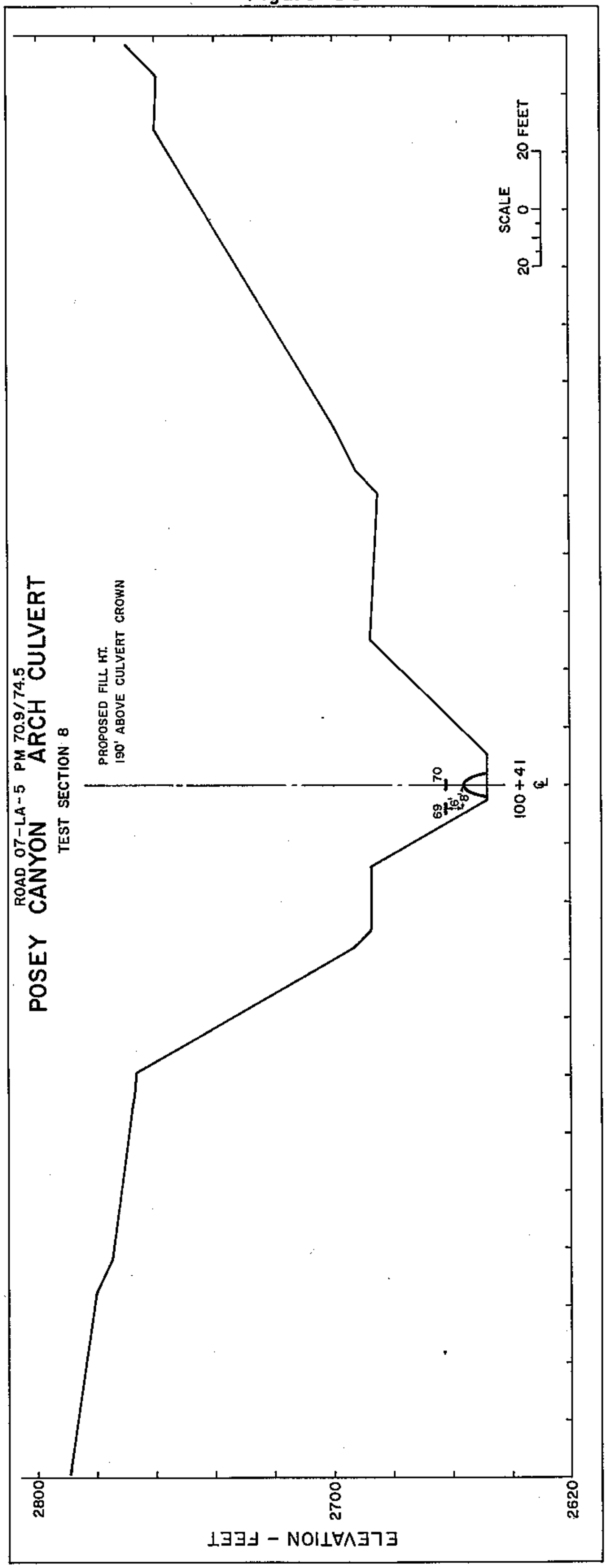


Figure 51

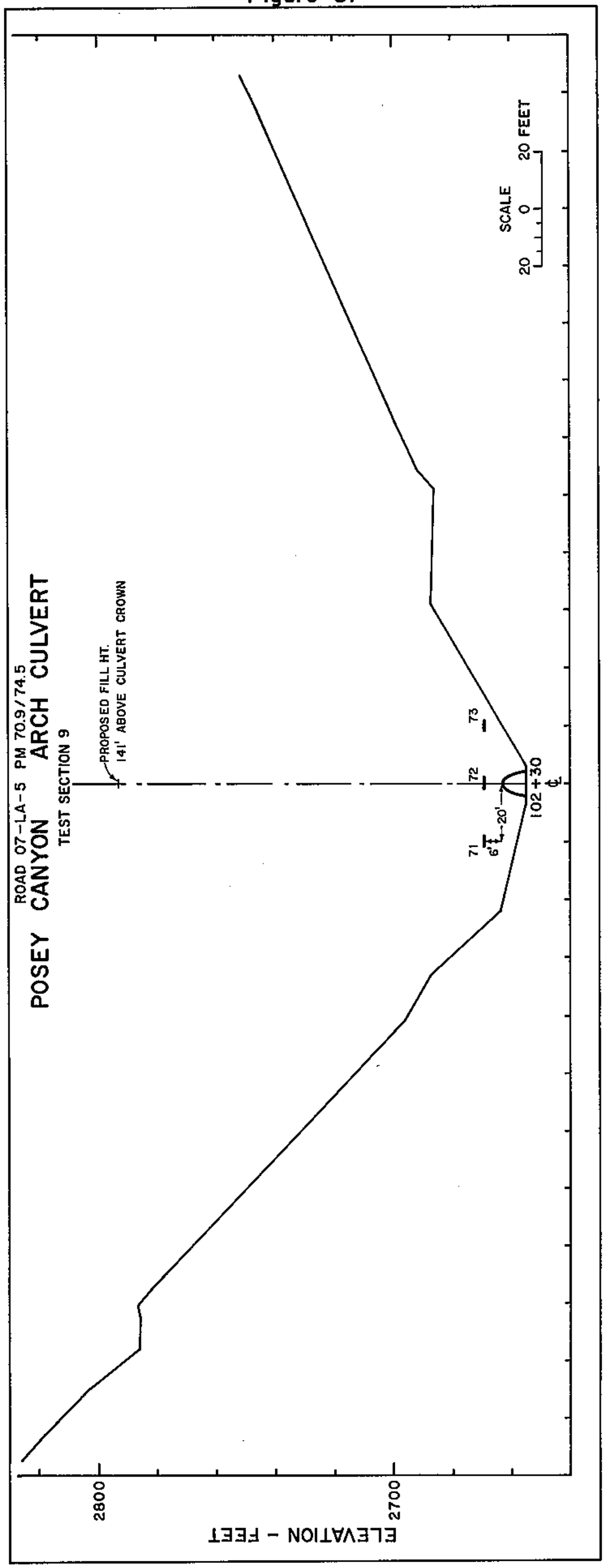
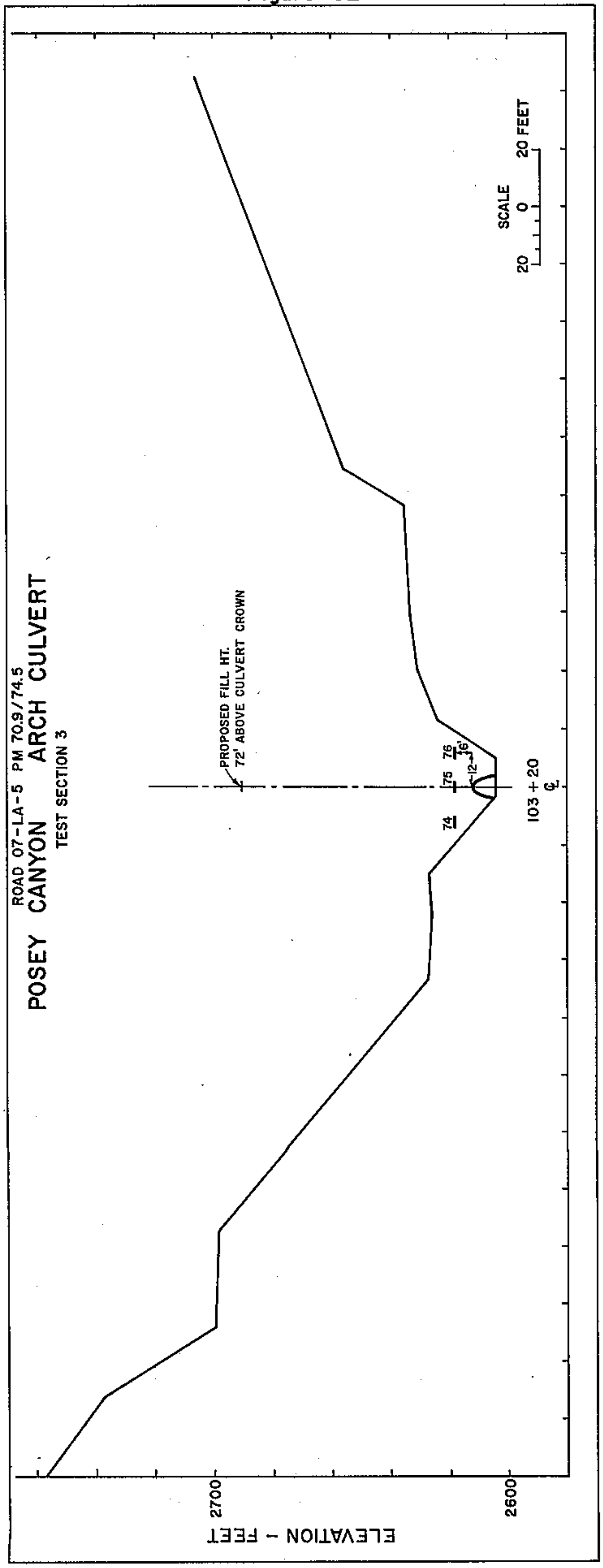


Figure 52


POSEY CANYON ARCH CULVERT
ROAD 07-LA-5 PM 70.9/74.5
TEST SECTION 3
PROPOSED FILL HT.
72' ABOVE CULVERT CROWN
74
75
76
12
16'
103 + 20
SCALE
20
0
20 FEET
2700
2600
ELEVATION - FEET